DESIGN CRITERIA FOR SEWAGE WORKS

1 AUGUST 1995

CHAPTER 7

Activated Sludge

ACTIVATED SLUDGE

7.1 General

7.1.1 Applicability

The activated sludge process and its various modifications may be used where sewage is amenable to biological treatment. This process requires close attention and more competent operator supervision than some of the other biological processes. A treatability study may be required to show that the organics are amendable to the proposed treatment. For example, industrial wastewaters containing high levels of starches and sugars may cause interferences with the activated sludge process due to bulking.

Toxic loadings from industries and excessive hydraulic loadings must be avoided to prevent the loss or destruction of the activated sludge mass. If toxic influents are a possibility, a properly enforced industrial pretreatment program will prove extremely beneficial to the WWTP and will be required. It takes days and sometimes weeks for the plant to recover from a toxic overload and will likely result in permit violations. Flow equalization, as detailed in Chapter 4, may be required in some instances. These requirements shall be considered when proposing this type of treatment.

7.1.2 Process Selection

The activated sludge process and its several modifications may be employed to accomplish varied degrees of removal of suspended solids and reduction of BOD and ammonia. Choice of the process most applicable will be influenced by the proposed plant size, type of waste to the treated, and degree and consistency of treatment required. All designs should provide for flexibility to incorporate as many modes of operation as is reasonably possible.

Calculations and/or documentation shall be submitted to justify the basis of design for the following:

a. Process efficiency
b. Aeration tanks
c. Aeration equipment (including oxygen and mixing requirements)
d. Operational rationale (including maintenance)
e. Costs (capital and operating)

In addition, the design must comply with any requirements set forth in other chapters such as clarifiers, sludge processing, etc.

7.1.3 Pretreatment

Where primary settling tanks are not used, effective removal or exclusion of grit, debris, excessive oil or grease, and comminution or screening of solids shall be accomplished prior to the activated sludge process.

Where primary settling is used, provisions should be made for discharging raw sewage directly to the aeration tanks to facilitate plant start-up and operation during the initial stages of the plant's design life. Also, primary effluents are often low in D.O. This should be planned for in the design.

7.2 Types of Processes

Figure 7.1 shows the flow schematics of the major types of activated sludge processes, excluding pretreatment. The types that are simply modifications of these processes are not shown.

7.2.1 Conventional

Conventional activated sludge is characterized by introduction of influent wastewater and return activated sludge at one end of the aeration tank, a plug-flow aeration tank, and diffused aeration.

7.2.2 Complete Mix

Complete mix activated sludge is characterized by introduction of influent wastewater and return activated sludge throughout the aeration basin and the use of a completely mixed aeration tank. Complete mix aeration tanks may be arranged in series to approximate plug flow and conventional activated sludge.

7.2.3 Step Aeration

Step aeration activated sludge is characterized by introduction of the influent wastewater at two or more points in the aeration tank, use of a plug-flow aeration tank, and diffused aeration.

7.2.4 Tapered Aeration

Tapered aeration is similar to conventional activated sludge except that the air supply is tapered to meet the organic load within the tank. More air is added to the influent end of the tank where the organic loading and oxygen demand are the greatest.

7.2.5 Contact Stabilization

Contact stabilization activated sludge is characterized by the use of two aeration tanks for each process train, one to contact the influent wastewater and return activated sludge (contact tank) and the other to aerate the return activated sludge (stabilization tank) and promote the biodegradation of the organics absorbed to the bacterial flocs.

7.2.6 Extended Aeration

Extended aeration activated sludge is characterized by a low F/M ratio, long sludge age, and long aeration tank detention time (greater than 18 hours). For additional details on oxidation ditches see Section 7.7).

7.2.7 High-Rate Aeration

High-rate aeration activated sludge is characterized by high F/M ratio, low sludge age, short aeration tank detention time, and high mixed-liquor suspended solids. High-rate aeration should be followed by other BOD and suspended solids removal processes to provide secondary treatment.

7.2.8 High-Purity Oxygen

High-purity oxygen activated sludge is characterized by the use of high-purity oxygen instead of air for aeration.

7.2.9 Kraus Process

Kraus process activated sludge is characterized by use of an aeration tank to aerate a portion of the return activated sludge, digester supernatant, and digested sludge in order to provide nitrogen (ammonia) to a nitrogen-deficient wastewater.

7.2.10 Sequencing Batch Reactors (SBR)

The SBR process is a fill-and-draw, non-steady state activated sludge process in which one or more reactor basins are filled with wastewater during a discrete time period, and then operated in a batch treatment mode. SBR's accomplish equalization, aeration, and clarification in a timed sequence. For additional details see Section 7.6.

7.3 Aeration Tanks

7.3.1 Required Volume

The size of the aeration tank for any particular adaptation of the process shall be based on the food-to-microorganism (F/M) ratio, using the influent BOD (load per day) divided by the mixed-liquor volatile suspended solids. Alternatively, aeration tanks may be sized using sludge age. The calculations using the F/M ratio or sludge age shall be based on the kinetic relationships.

APPENDIX 7A shows the permissible range of F/M ratio, sludge age, mixed-liquor suspended solids, aeration tank detention time, aerator loading, and activated sludge return ratio for design of the various modifications of the activated sludge process. All design parameters shall be checked to determine if they fall within the permissible range for the selected F/M ratio or sludge area and the aeration tank size. Diurnal load variations and peak loadings must be considered when checking critical parameters.

7.3.2 Shape and Mixing

The dimensions of each independent mixed-liquor aeration tank or return sludge reaeration tank should be such as to maintain effective mixing and utilization of air when diffused air is used. Liquid depths should not be less than 10 feet or more than 30 feet except in special design cases. For plug-flow conditions using very small tanks or tanks with special configuration, the shape of the tank and/or the installation of aeration equipment should provide for elimination of short-circuiting through the tank.

Aerator loadings should be considered and the horsepower per 1,000 cubic feet of basin volume required for oxygen transfer should be limited to prevent excessive turbulence in the aeration basins, which might reduce activated sludge settleability.

7.3.3 Number of Units

Multiple tanks capable of independent operation may be required for operability and maintenance reasons, depending on the activated sludge process, size of the plant, and the reliability classification of the sewerage works (refer to Section 1.3.11).

7.3.4 Inlets and Outlets

7.3.4.1 Controls

Inlets and outlets for each aeration tank unit in multiple tank systems should be suitably equipped with valves, gates, stop plates, weirs, or other devices to permit control of the flow and to maintain reasonably constant liquid level. The hydraulic properties of the system should permit the maximum instantaneous hydraulic load to be carried with any single aeration tank unit out of service.

7.3.4.2 Conduits

Channels and pipes carrying liquids with solids in suspension should be designed to maintain self-cleaning velocities or should be agitated to keep such solids in suspension at all rates of flow within the design limits.

7.3.4.3 Hydraulics

Where multiple aeration tanks and secondary clarifiers are used, provisions should be made to divide the flow evenly to all aeration tanks in service and then recombine the flows, and to divide the flow evenly to all secondary clarifiers in service and then recombine the flows. Treatments plants using more than four aeration tanks and secondary clarifiers may divide the activated sludge systems into two or more process trains consisting of not less than two aeration tanks and secondary clarifiers per process train.

7.3.4.4 Bypass

When a primary settling tank is used, provisions shall also be made for discharging raw wastewater directly to the aeration tanks following pretreatment for start-ups.

7.3.5 Measuring Devices

For plants designed for less than 250,000 gallons per day, devices shall be installed for indicating flow rates of influent sewage, return sludge, and air to each aeration tank. For plants designed for greater than 250,000 gallons per day, devices shall be installed for totalizing, indicating, and recording influent sewage and returned sludge to each aeration tank. Where the design provides for all returned sludge to be mixed with the raw sewage (or

primary effluent) at one location, the mixed-liquor flow rate to each aeration tank shall be measured, and the flow split in such a manner to provide even loading to each tank, or as desired by operations.

7.3.6 Freeboard and Foam Control

Aeration tanks shall have a freeboard of at least 18 inches. Freeboards of 24 inches are desirable with mechanical aerators.

Consideration shall be given for foam control devices on aeration tanks. Suitable spray systems or other appropriate means will be acceptable. If potable water is used, approved backflow prevention shall be provided on the water lines. The spray lines shall have provisions for draining to prevent damage by freezing.

7.3.7 Drain and Bypass

Provisions shall be made for dewatering each aeration tank for cleaning and maintenance. The dewatering system shall be sized to permit removal of the tank contents within 24 hours. If a drain is used, it shall be valved. The dewatering discharge shall be upstream of the activated sludge process.

Provisions shall be made to isolate each aeration tank without disrupting flow to other aeration tanks.

Proper precautions shall be taken to ensure the tank will not "float" when dewatered.

7.3.8 Other Considerations

Other factors that might influence the efficiency of the activated sludge process should be examined. Septic and/or low pH influent conditions are detrimental, particularly where primary clarifiers precede the activated sludge process or when the collection system allows the sewage to go septic. Often, the pH is buffered by the biological mass, but wide variations in the influent should be avoided and, if present, chemical addition may be necessary.

Aerobic organisms require minimum quantities of nitrogen and phosphorus. Domestic wastewater usually has an excess of nitrogen and phosphorus; however, many industrial wastewaters are deficient in these elements. A mass balance should be performed to see if the combined industrial and domestic influent contains sufficient nitrogen and phosphorus or if nutrient levels will have to be supplemented.

7.4 Aeration Equipment

7.4.1 General

Oxygen requirements generally depend on BOD loading, degree of treatment, and level of suspended solids concentration to be maintained in the aeration tank mixed liquor. Aeration equipment shall be designed to supply sufficient oxygen to maintain a minimum dissolved oxygen concentration of 2 milligrams per liter (mg/l) at average design load and 1.0 mg/l at peak design loads throughout the mixed liquor. In the absence of experimentally determined values, the design oxygen requirements for all activated sludge processes shall be 1.1 lbs oxygen per lb peak BOD_5 applied to the aeration tanks, with the exception of the extended aeration process, for which the value shall be 2.35. Aeration equipment shall be of sufficient size and arrangement to maintain velocities greater than 0.5 foot per second at all points in the aeration tank.

The oxygen requirements for an activated sludge system can be estimated using the following relationship:

O_2 = (a) (BOD) + b (MLVSS)

O_2 = pounds of oxygen required per day

BOD = pounds of BOD removed per day (5-day BOD)*

MLVSS= pounds of mixed liquor volatile suspended solids contained in the aeration basin

a = amount of oxygen required for BOD synthesis. "a" will range from 0.5 to 0.75 pound of oxygen per pound of BOD removed

b = amount of oxygen required for endogenous respiration or decay. "b" will range from 0.05 to 0.20 pound of oxygen per pound of MLVSS

*BOD removal shall be calculated as influent BOD_5 minus soluble effluent BOD_5.

For preliminary planning before process design is initiated, a rough estimate can be obtained by using 1.0 to 1.2 pounds of oxygen per pound of BOD removed (assuming no nitrification).

7.4.2 Diffused Air Systems

7.4.2.1 Design Air Requirements

The aeration equipment shall be designed to provide the oxygen requirements set forth above. Minimum requirements for carbonaceous removal are shown below. (Oxygen requirements for nitrification are in addition to that required for carbonaceous removal where applicable; i.e., low F/M.)

Process	Cubic Feet of Air Available per Pound of BOD Load Applied to Aeration Tank
Conventional	1,500
Step Aeration	1,500
Contact Stabilization	1,500
Modified or "High Rate"	400 to 1,500 (depending upon BOD removal expected)
Extended Aeration	2,100

Air required for channels, pumps, or other air-use demand shall be added to the air volume requirements.

Manufacturers' specifications must be corrected to account for actual operation conditions (use a worst case scenario). Corrections shall be made for
temperatures other than 20^{o}C and elevations greater than 2,000 feet.

7.4.2.2 Special Details

The specified capacity of blowers or air compressors, particularly centrifugal blowers, shall take into account that the air intake temperature might reach extremes and that pressure might be less than normal. Motor horsepower shall be sufficient to handle the minimum and maximum ambient temperatures on record.

The blower filters shall be easily accessible. Spare filters should be provided.

The blowers shall be provided in multiple units, arranged and in capacities to meet the maximum air demand with the single largest unit out of service. The design shall also provide for varying the volume of air delivered in proportion to the load demand of the plant.

The spacing of diffusers shall be in accordance with the oxygen and mixing requirements in the basin. If only one aeration tank is proposed, arrangement of diffusers should permit their removal for inspection, maintenance, and replacement without de-watering the tank and without shutting off the air supply to other diffusers in the tank.

Individual units of diffusers shall be equipped with control valves, preferably with indicator markings, for throttling or for complete shutoff. Diffusers in each assembly shall have substantially uniform pressure loss. The adjustment of one diffuser should have minimal influence on the air supply rate to any other diffusers.

Flow meters and throttling valves shall be placed in each header. Air filters shall be provided as part of the blower assembly to prevent clogging of the diffuser system. Means shall be provided to easily check the air filter so that it will be replaced when needed.

7.4.3 Mechanical Aeration Equipment

Power input from mechanical aerators should range from 0.5 to 1.3 horsepower per 1,000 cubic feet of aeration tank.

The mechanism and drive unit shall be designed for the expected conditions of the aeration tank in terms of the proven performance of the equipment.

Due to the high heat loss, consideration shall be given to protecting subsequent treatment units from freezing where it is deemed necessary. Multiple mechanical aeration unit installations shall be designed to meet the maximum oxygen demand with the largest unit out of service. The design shall normally also provide for varying the amount of oxygen transferred in proportion to the load demand on the plant.

A spare aeration mechanism shall be furnished for single-unit installations. Access to the aerators shall be provided for routine maintenance.

7.4.4 Flexibility and Energy Conservation

The design of aeration systems shall provide adequate flexibility to vary the oxygen transfer capability and power consumption in relation to oxygen demands. Particular attention should be given to initial operation when oxygen demands may be significantly less that the design oxygen demand. The design shall always maintain the minimum mixing levels; mixing may control power requirements at low oxygen demands.

Dissolved oxygen probes and recording should be considered for all activated sludge designs. Consideration will be given to automatic control of aeration system oxygen transfer, based on aeration basin dissolved oxygen concentrations, provided manual back-up operation is available. A dissolved oxygen field probe and meter is to be provided for all activated sludge installations.

Watt-hour meters shall be provided for all aeration system drives to record power usage.

Energy conservation measures shall be considered in design of aeration systems. For diffused aeration systems, the following shall be considered:

a. Use of small compressors and more units

b. Variable-speed drives on positive-displacement compressors

c. Intake throttling on centrifugal compressors

d. Use of timers while maintaining minimum mixing and D.O. levels (consult with manufacturer's recommendations for proper cycling)

e. Use of high-efficiency diffusers

f. Use of separate and independent mixers and aerators

For mechanical aeration systems, the following shall be considered:

a. Use of smaller aerators

b. Variable aeration tank weirs

c. Multiple-speed motors

d. Use of timers

7.5 Additional Details

7.5.1 Lifting Equipment and Access

Provisions shall be made to lift all mechanical equipment and provide sufficient access to permit its removal without modifying existing or proposed structures.

7.5.2 Noise and Safety

Special consideration shall be given to the noise produced by air compressors used with diffused aeration systems and mechanical aerators. Ear protection may be required. Silencers for blowers may be required in sensitive areas.

Handrails shall be provided on all walkways around aeration tanks and clarifiers.

The following safety equipment shall be provided near aeration tanks and clarifiers:

Safety vests
Lifelines and rings
Safety poles

Walkways near aeration tanks shall have a roughened surface or grating to provide safe footing and be built to shed water.

Guards shall be provided on all moving machinery in conformance with OSHA requirements.

Sufficient lighting shall be provided to permit safe working conditions near aerations tanks and clarifiers at night.

7.6 Sequencing Batch Reactors (SBRs)

SBRs shall be designed to meet all the requirements set forth in preceding sections on activated sludge. Special consideration shall be given to the following:

7.6.1 A pre-aeration, flow-equalization basin is to be provided for when the SBR is in the settle and/or draw phases. If multiple SBR basins are provided, a pre-aeration basin will not be needed if each SBR basin is capable of handling all the influent peak flow while another basin is in the settle and/or draw phase.

7.6.2 When discharging from the SBR, means need to be provided to avoid surges to the succeeding treatment units. The chlorine contact tank shall not be hydraulically overloaded by the discharge.

7.6.3 The effluent from the SBRs shall be removed from just below the water surface (below the scum level) or a device which excludes scum shall be used. All decanters shall be balanced so that the effluent will be withdrawn equally from the effluent end of the reactor.

7.6.4 Prevailing winds must be considered in scum control.

7.7 Oxidation Ditch

7.7.1 General

The oxidation ditch is a complete-mixed, extended aeration, activated sludge process which is operated with a long detention time. Brush-rotor (or disk type) aerators are normally used for mixing and oxygen transfer. All requirements set forth in previous sections and/or chapters must be met, with the exception of those items addressed below.

7.7.2 Special Details

7.7.2.1 Design Parameters

The design parameters shall be in the permissible range as set forth in Table 7.1 for F/M, sludge age, MLSS, detention time, aerator loading, and activated sludge return ratio.

7.7.2.2 Aeration Equipment

Aeration equipment shall be designed to transfer 2.35 pounds of oxygen per pound of BOD at standard conditions. The oxygen requirement takes into account nitrification in a typical wastewater. Also, a minimum average velocity of one foot per second shall be maintained, based on the pumping rate of the aeration equipment and the aeration basin cross-sectional area.

A minimum of two aerators per basin is required.

7.7.2.3 Aeration Tank Details

a. Influent Feed Location

Influent and return activated sludge feed to the aeration tank should be located just
upstream of an aerator to afford immediate mixing with mixed liquor in the channel.

b. Effluent Removal Location

Effluent from the aeration channel shall be upstream of an aerator and far enough upstream from the injection of the influent and return activated sludge to prevent short-circuiting.

c. Effluent Adjustable Weir

Water level in the aeration channel shall be controlled by an adjustable weir or other means. In calculating weir length, use peak design flow plus maximum recirculated flow to prevent excessive aerator immersion.

d. Walkways and Splash Control

Walkways must be provided across the aeration channel to provide access to the aerators for maintenance. The normal location is above the aerator. Splash guards shall be provided to prevent spray from the aerator on the walkway. Bridges should not be subject to splash from the rotors.

e. Baffles

Horizontal baffles, placed across the channel, may be used on all basins with over 6 feet liquid depth, and may be used where the manufacturer recommends them to provide proper mixing of the entire depth of the basin.

Baffles should be provided around corners to ensure uniform velocities.

7.7.3 45-Degree Sloping Sidewall Tanks

7.7.3.1 Liquid Depth

Liquid depth shall be 7 to 10 feet, depending on aerator capability, as stated by the manufacturer.

7.7.3.2 Channel Width at Water Level

The higher ratios (channel width at water level divided by aerator length) are to be used with smaller aerator lengths.

3- to 15-foot-long rotors, ratio 3.0 to 1.8.

16- to 30-foot-long rotors, ratio 2.0 to 1.3

Above 30-foot-long rotors, ratio below 1.5

7.7.3.3 Center Island

When used, the minimum width of center island at liquid level, based on aerator length, should be as follows (with center islands below minimum width, use return flow baffles at both ends):

3- to 5-foot-long rotor, 14 feet

6- to 15-foot-long rotor, 16 feet

16- to 30-foot-long rotor, 20 feet

Above 30-foot-long rotors, 24 feet

7.7.3.4 Center Dividing Walls

Center dividing walls can be used but return flow baffles at both ends are required. The channel width, W, is calculated as flat bottom plus 1/2 of sloping sidewall. Baffle radius is W/2. Baffles should be offset by W/8, with the larger opening accepting the flow and the smaller opening downstream compressing the flow.

7.7.3.5 Length of Straight Section

Length of straight section of ditch shall be a minimum of 40 feet or at least two times the width of the ditch at liquid level.

7.7.3.6 Preferred Location of Aerators

Aerators shall be placed just downstream of the bend, normally 15 feet, with the long straight section of the ditch downstream of the aerator.

7.7.4 Straight Sidewall Tanks

7.7.4.1 Liquid Depth

Liquid depth shall be 7 to 12 feet, depending on aerators.

7.7.4.2 Aerator Length

Individual rotor length shall span the full width of the channel, with necessary allowances required for drive assembly and outboard bearing.

7.7.4.3 Center Island

Where center islands are used, the width should be the same as with 45-degree sloping sidewalls, or manufacturer's recommendation.

7.7.4.4 Center Dividing Walls

When a center dividing wall is used, return flow baffles are required at both ends. Return flow baffle radius is width of channel, W, divided by 2, W/2. Baffles should be offset by W/8, with the larger opening accepting the flow and the smaller opening downstream compressing the flow.

7.7.4.5 Length of Straight Section

Length of straight section downstream of aerator shall be near 40 feet or close to two times the aerator length. In deep tanks with four aerators, aerators should be placed to provide location for horizontal baffles.

7.7.4.6 Preferred Location of Aerators

Aerators should be placed just downstream of the bend with the long straight section of the tank downstream of the aerator. Optimal placement of rotors will consider maintaining ditch center line distance between rotors close to equal.

CHAPTER 8

Nitrification

8.1 General

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NITRIFICATION

8.1 General

8.1.1 Applications

Nitrogen exists in treated wastewater primarily in the form of ammonia which is oxidized to nitrate by bacteria. This process requires oxygen and can exert a significant oxygen demand on the receiving water.

Nitrification shall be considered when ammonia concentrations in the effluent would cause the receiving water to exceed the limitations established to prevent ammonia toxicity to aquatic life, or when the effluent ammonia quantity would cause the dissolved oxygen level of the receiving stream to deplete below allowable limits. The degree of treatment required will be determined by the NPDES permit limit.

8.1.2 Process Selection

Calculations shall be submitted to support the basis of design. The following factors should be considered in the evaluations of alternative nitrification processes:

a. Ability to meet effluent requirements under all environmental conditions to be encountered, with special emphasis on temperature, pH, alkalinity, and dissolved oxygen.

b. Cost (total present worth)

c. Operational considerations, including process stability, flexibility, operator skill required, and compatibility with other plant processes.

d. Land requirements.

8.2 Suspended Growth Systems

8.2.1 Single - Stage Activated Sludge

This section details the requirements for activated sludge systems designed to both remove carbonaceous matter and oxidize ammonia.

8.2.1.1 Process Design

Design must provide adequate solids retention time in the activated sludge system for sufficient growth of nitrifying bacteria. A safety factor of 2.5 or greater should be used to calculate the design mean cell residence time or sludge age. This safety factor must be large enough to provide enough operational flexibility to handle diurnal, peak, and transient loadings. The calculation of the solids retention time shall consider influent BOD, TSS, BOD_5/TKN (Total Kjeldahl Nitrogen) ratio and kinetic parameters. The kinetic parameters can be taken from the literature, similar installations, or pilot plant studies. The effect of temperature on the kinetics must be considered since nitrification will not proceed as rapidly during winter months.

8.2.1.2 Special Details

The following requirements are in addition to those included in Chapter 5, "Clarifiers", and Chapter 7, "Activated Sludge":

a. Sufficient oxygen must be provided for both carbonaceous BOD oxidation and ammonia oxidation. Use 4.6 pounds O_2 per pound total Kjeldahl nitrogen to calculate the oxygen requirements for nitrification, in addition to the oxygen needed for BOD removal.

b. Aeration basin design dissolved oxygen shall be greater than or equal to 2.0 mg/l.

c. Diurnal peak mass flow rates of BOD and total Kjeldahl nitrogen must be considered in the aeration system design.

d. The pH levels must be controlled within the range of 6.5 to 8.4. Nitrification is optimized in the upper portion of this range (7.9 to 8.4) but pH levels in the range of 7.6 to 7.8 are recommended since CO_2 produced will be released from the wastewater.

e. Nitrification requires alkalinity, 7.1 pounds as $CaCO_3$ per pound NH_3-N oxidized. The wastewater must be shown to have sufficient alkalinity or chemical treatment must be considered to provide adequate alkalinity.

f. Clarifier and return sludge pumping must be designed with the capability to allow operation over a range of solids retention times. Flexibility should be provided to prevent denitrification in the clarifier from low D.O. levels in the sludge blanket. This could cause violations of other effluent limits (i.e., suspended solids).

8.2.2 Two-Stage with Activated Sludge Nitrification

This section details the requirements for systems in which carbonaceous BOD is removed in the first stage and ammonia is oxidized by activated sludge in the second stage. BOD removal in the first stage could be by activated sludge, trickling filters, or physical - chemical treatment.

8.2.2.1 Process Design

The first stage shall be designed using the requirements of the appropriate chapters, such as activated sludge, trickling filters, and clarifiers. To promote a sludge with good settling characteristics in the second stage clarifier, some carbonaceous BOD shall enter the second stage aeration basin. This allows a less conservative design of the first stage as long as total BOD removal is sufficient. The requirements for the process design of the second stage are the same as those presented previously for the single-stage nitrification system.

8.2.2.2 Special Details

The following details are in addition to those in Chapter 5, "Clarifiers," Chapter 6, "Fixed Film Reactors," and Chapter 7, "Activated Sludge."

a. Sufficient oxygen must be provided for both carbonaceous BOD oxidation and ammonia oxidation. Use 4.6 pounds O_2 per pound total Kjeldahl nitrogen to calculate the oxygen requirements for nitrification, in addition to the oxygen needed nitrogen to calculate the oxygen requirements for nitrification, in addition to the oxygen needed for BOD removal.

b. Aeration basin design dissolved oxygen shall be greater than or equal to 2.0 mg/l.

c. Diurnal peak mass flow rates of BOD and total Kjeldahl nitrogen must be considered in the aeration system design.

d. The pH levels must be controlled within the range of 6.5 to 8.4. Nitrification is optimized in the upper portion of this range (7.9 to 8.4) but pH levels in the range of 7.6 to 7.8 are recommended since CO_2 produced will be released from the wastewater.

e. Nitrification requires alkalinity, 7.1 pounds as $CaCO_3$ per pound NH_3-N oxidized. The wastewater must be shown to have sufficient alkalinity or chemical treatment must be considered to provide adequate alkalinity.

f. Clarifier and return sludge pumping must be designed with the capability to allow operation over a range of solids retention times. Flexibility should be provided to prevent denitrification in the clarifier from low D.O. levels in the sludge blanket. This could cause violations of other effluent limits (i.e., suspended solids).

8.3 Fixed - Film Systems

8.3.1 Trickling Filters

8.3.1.1 Process Design

Recirculation is required to provide a constant hydraulic loading on the medium.

a. Single - Stage

This section details the requirements for a trickling filter that is designed for both carbonaceous BOD removal and ammonia oxidation. Design shall be based on the organic loading expressed as pounds BOD per 1,000 cubic feet. The design loading rate shall by justified from literature, similar installations, or pilot plant data for a particular depth and type of filter medium. Design shall consider temperature effects on ammonia removal and organic loading rates, and any proposal to attain nitrification in a single-stage rock media trickling filter will be more closely scrutinized than with other types of media.

b. Two - Stage

This section details the requirements of using a trickling filter for nitrification which is preceded by a trickling filter, activated sludge system, or physical - chemical

treatment for carbonaceous BOD removal. Design must be based on either a surface area loading expressed as square feet per pound NH_4-N oxidized per day or a volumetric loading expressed as pounds NH_4-N per 1,000 cubic feet per day. Loading rates must be justified from literature, similar plants, or
pilot plant data. The effects of temperature on loading rates and ammonia oxidation must be considered in the design.

8.3.1.2 Special Details

The following requirements are in addition to those in Chapter 5, "Clarifiers," and Chapter 6, "Fixed Film Reactors."

a. Clarifiers will be required for second-stage trickling filters for nitrification.

b. Higher specific surface area and lower void ratio media may be used for second-stage trickling filters providing nitrification.

8.3.2 Activated Biofilter (ABF) Process

8.3.2.1 Process Design

Process design shall be based on the literature, similar installations, or pilot plant data. The design shall consider the effects of temperature, pH, and aeration basins.

8.3.2.2 Special Details

a. Sufficient oxygen must be provided for both carbonaceous BOD oxidation and ammonia oxidation. Use 4.6 pounds O_2 per pound total Kjeldahl nitrogen to calculate the oxygen requirement for nitrification, in addition to the oxygen needed for BOD removal.

b. Aeration basin design dissolved oxygen shall be greater than or equal to 2.0 mg/l.

c. Diurnal peak mass flow rates of BOD and total Kjeldahl nitrogen must be considered in the aeration system design.

d. The pH levels must be controlled within the range of 6.5 to 8.4. Nitrification is optimized in the upper portion of this range (7.9 to 8.4) but pH levels in the range of 7.6 to 7.8 are recommended since CO_2 produced will be released from the wastewater.

e. Nitrification requires alkalinity, 7.1 pounds as $CaCO_3$ per pound NH_3-N oxidized. The wastewater must be shown to have sufficient alkalinity or chemical treatment must be considered to provide adequate alkalinity.

f. Clarifier and return sludge pumping must be designed with the capability to allow operation over a range of solids retention times. Flexibility should be provided to prevent denitrification in the clarifier from low D.O. in the sludge blanket. This could cause violations of other effluent limits (i.e., suspended solids).

8.3.3 Submerged Media

8.3.3.1 General

This section includes all designs for fixed-film reactors using stones, gravel, sand, anthracite coal, or plastic media or combinations thereof in which the medium is submerged and air or oxygen is used to maintain aerobic conditions. Pilot plant testing or a similar full-scale installation with a minimum of 1 year of operation is required before consideration will be given to a submerged design. No design will be considered unless the following can be demonstrated:

a. Reliable operation

b. Ability to transfer sufficient oxygen

c. Ability to handle peak flows without washout of medium

d. Methods of separating suspended solids from effluent, removing waste sludge, and stabilization and dewatering of waste sludge

e. Media resistance to plugging

8.3.3.2 Process Design

Data for design and calculations shall be submitted upon request to justify the basis of design.

8.3.4 Rotating Biological Contactors

8.3.4.1 Process Design

Process design shall be based on the surface area loading expressed as gallons per day per square foot. Design surface area loading shall consider the number of stages, temperature, BOD concentration entering and leaving each stage, and ammonia concentration entering and leaving each stage. Calculations shall be submitted upon request to justify the basis of design.

8.3.4.2 Special Details

The following requirements are in addition to those set forth in Chapter 5, "Clarifiers," and Chapter 6, "Fixed Film Reactors."

a. Standard media (100,000 square feet per shaft or less) shall be used until influent BOD concentration is less than manufacturer's recommendation for high-density media (150,000 square feet per shaft or more). High-density media may be used for influent BOD concentrations less than manufacturer's recommendation for high-density media.

b. Clarifiers will be required following rotating biological contactors that follow a secondary process.

CHAPTER 9

Ponds and Aerated Lagoons

PONDS AND AERATED LAGOONS

9.1 General

This chapter describes the requirements for the following biological treatment processes:

a. Stabilization ponds

b. Aerated lagoons

Additionally, this chapter describes the requirements for use of hydraulic control release lagoons for effluent disposal.

A guide to provisions for lagoon design is the EPA publication Design Manual - Municipal Wastewater Stabilization Ponds, EPA-625/1-83-015.

9.1.1 Applicability

In general, ponds and aerated lagoons are most applicable to small and/or rural communities where land is available at low cost and minimum secondary treatment requirements are acceptable. Advantages include potentially lower capital costs, simple operation, and low O&M costs.

9.1.2 Supplement to Engineering Report

The engineering report shall contain pertinent information on location, geology, soil conditions, area for expansion, and any other factors that will affect the feasibility and acceptability of the proposed treatment system.

The following information should be submitted in addition to that required in the Chapter 1 section titled "Engineering Report and Preliminary Plans":

a. The location and direction of all residences, commercial development, and water supplies within 1/2 mile of the proposed pond

b. Results of the geotechnical investigation performed at the site

c. Data demonstrating anticipated seepage rates of the proposed pond bottom at the maximum water surface elevation

d. A description, including maps showing elevations and contours, of the site and adjacent area suitable for expansion

e. The ability to disinfect the discharge is required.

9.1.3 Effluent Requirements

See Chapter 1, Section 1.1.

9.2 Design Loadings

9.2.1 Stabilization Ponds

Stabilization ponds are facultative and are not artificially mixed or aerated. Mixing and aeration are provided by natural processes. Oxygen is supplied mainly by algae.

Design loading shall not exceed 30 pounds BOD per acre per day on a total pond area basis and 50 pounds BOD per acre per day to any single pond (from Middlebrooks).

9.2.2 Aerated Lagoons

An aerated lagoon may be a complete-mix lagoon or a partial-mix aerated lagoon. Complete-mix lagoons provide enough aeration or mixing to maintain solids in suspension. Power levels are normally between 20 and 40 horsepower per million gallons. The partial-mix aerated lagoon is designed to permit accumulation of settleable solids on the lagoon bottom, where they decompose anaerobically. The power level is normally 4 to 10 horsepower per million gallons of volume.

BOD removal efficiencies normally vary from 80 to 90 percent, depending on detention time and provisions for suspended solids removal.

The aerated lagoon system design for minimum detention time may be estimated by using the following formula; however, for the development of final parameters, it is recommended that actual experimental data be developed.

$$\frac{Se}{So} = \frac{1}{1 + 2.3K_1 t}$$

where:

t = detention time, days
K_1= reaction coefficient, complete system per day, base 10. For complete treatment of normal domestic sewage, the K_1 value will be assumed to be:
$K_1 = 1.087$ @20^oC for complete mix
$K_1 = 0.12$ @20^oC for partial mix
Se = effluent BOD_5, mg/l
So = influent BOD_5, mg/l

The reaction rate coefficient for domestic sewage that includes significant quantities of industrial wastes, other wastes, and partially treated sewage should be determined experimentally for various conditions that might be encountered in the aerated ponds. Conversion of the reaction rate coefficient to temperatures other than 20 degrees C should be according to the following formula:

$K_1 = K_{20}\ 1.036^{(T-20)}$ (T = temperature in degrees C)

The minimum equilibrium temperature of the lagoon should be used for design of the aerated lagoon. The minimum equilibrium temperature should be estimated by using heat balance equations, which should include factors for influent wastewater temperature, ambient air temperature, lagoon surface area, and heat transfer effects of aeration, wind, and humidity. The minimum 30-day average ambient air temperature obtained from climatological data should be used for design.

Additional storage volume shall be considered for sludge storage and partial mix in aerated lagoons.

Sludge processing and disposal should be considered.

9.3 Special Details

9.3.1 General

9.3.1.1 Location

a. Distance from Habitation

A pond site should be located as far as practicable from habitation or any area that may be built up within a reasonable future period, taking into consideration site specifics such as topography, prevailing winds, and forests. Buffer zones between the lagoon and residences or similar land use should be at least 300 feet to residential property lines, and 1000 feet to existing residence structures.

b. Prevailing Winds

If practical, ponds should be located so that local prevailing winds will be in the direction of uninhabited areas. Preference should be given to sites that will permit an unobstructed wind sweep across the length of the ponds in the direction of the local prevailing winds.

c. Surface Runoff

Location of ponds in watersheds receiving significant amounts of runoff water is discouraged unless adequate provisions are made to divert storm water around the ponds and protect pond embankments from erosion.

d. Water Table

The effect of the ground water location on pond performance and construction must be considered.

e. Ground Water Protection

Ground Water Protection's main emphasis should be on site selection and liner construction, utilizing mainly compacted clay. Proximity of ponds to water supplies and other facilities subject to contamination and location in areas of porous soils and fissured rock formations should be critically evaluated to avoid creation of health hazards or other undesirable conditions. The possibility of chemical pollution may merit appropriate consideration. Test wells to monitor potential ground water pollution may be required and should be designed with proper consideration to water movement through the soil as appropriate.

An approved system of ground water monitoring wells or lysimeters may be required around the perimeter of the pond site to facilitate ground water monitoring. The use of wells and/or lysimeters will be determined on a case-by-case basis depending on proximity of water supply and maximum ground water levels. This determination will be at the site approval phase (see Section 1.1).

A routine ground water sampling program shall be initiated prior to and during the pond operation, if required.

f. Floodwaters

Pond sites shall not be constructed in areas subject to 25-year flooding, or the ponds and other facilities shall be protected by dikes from the 25-year flood.

9.3.1.2 Pond Shape

The shape of all cells should be such that there are no narrow or elongated portions. Round, square, or rectangular ponds should have a length to width ratio near 1:1 for complete mix ponds. Rectangular ponds with a length <u>not exceeding three times the width</u> are considered most desirable for <u>complete mix</u> aerated lagoons. <u>However, stabilization ponds</u> should be rectangular with a <u>length exceeding three times the width,</u> or be baffled to ensure full utilization of the basin. No islands, peninsulas, or coves are permitted. Dikes should be rounded at corners to minimize accumulations of floating materials. Common dike construction should be considered whenever possible to minimize the length of exterior dikes.

9.3.1.3 Recirculation

Recirculation of lagoon effluent may be considered. Recirculation systems should be designed for 0.5 to 2.0 times the average influent wastewater flow and include flow measurement and control.

9.3.1.4 Flow Measurement

The design shall include provisions to measure, total, and record the wastewater flows.

9.3.1.5 Level Gauges

Pond level gauges should be located on outfall structures or be attached to stationary structures for each pond.

9.3.1.6 Pond Dewatering

All ponds shall have emergency drawdown piping to allow complete draining for maintenance.

Sufficient pumps and appurtenances should be available to facilitate draining of individual ponds in cases where multiple pond systems are constructed at the same elevation or for use if recirculation is desired.

9.3.1.7 Control Building

A control building for laboratory and maintenance equipment should be provided.

9.3.1.8 General Site Requirements

The pond area shall be enclosed with an adequate fence to keep out livestock and discourage trespassing, and be located so that travel along the top of the dike by maintenance vehicles is not obstructed. A vehicle access gate of width sufficient to accommodate mowing equipment and maintenance vehicles should be provided. All access gates shall be provided with locks.

Cyclone-type fences, 5 to 6 feet high with 3 strands of barbed wire, are desirable, with appropriate warning signs required.

9.3.1.9 Provision for Sludge Accumulation

Influent solids, bacteria, and algae that settle out in the lagoons will not completely decompose and a sludge blanket will form. This can be a problem if the design does not include provisions for removal and disposal of accumulated sludge, particularly in the cases of anaerobic stabilization ponds and aerated lagoons. The design should include an estimate of the rate of sludge accumulation, frequency of sludge removal, methods of sludge removal, and ultimate sludge handling and disposal. Abandoning and capping of the lagoon is an acceptable solution (Re: The Division of Solid Waste Management guidelines for abandonment of a lagoon). However, the design life shall be stated in the report.

9.3.2 Stabilization Ponds

9.3.2.1 Depth

The primary (first in a series) pond depth should not exceed 6 feet. Greater depths will be considered for polishing ponds and the last ponds in a series of 4 or more.

9.3.2.2 Influent Structures and Pipelines

a. Manholes

A manhole should be installed at the terminus of the interceptor line or the force main and should be located as close to the dike as topography permits; its invert should be at least 6 inches above the maximum operating level of the pond to provide sufficient hydraulic head without surcharging the manhole.

b. Influent Pipelines

The influent pipeline can be placed at zero grade. The use of an exposed dike to carry the influent pipeline to the discharge points is prohibited, as such a structure will impede circulation.

c. Inlets

Influent and effluent piping should be located to minimize short-circuiting and stagnation within the pond and maximize use of the entire pond area.

Multiple inlet discharge points shall be used for primary cells larger than 10 acres.

All gravity lines should discharge horizontally onto discharge aprons. Force mains should discharge vertically up and shall be submerged at least 2 feet when operating at the 3-foot depth.

d. Discharge Apron

Provision should be made to prevent erosion at the point of discharge to the pond.

9.3.2.3 Interconnecting Piping and Outlet Structures

Interconnecting piping for pond installations shall be valved or provided with other arrangements to regulate flow between structures and permit variable depth control.

The outlet structure can be placed on the horizontal pond floor adjacent to the inner toe of the dike embankment. A permanent walkway from the top of the dike to the top of the outlet structure is required for access.

The outlet structure should consist of a well or box equipped with multiple-valved pond drawoff lines. An adjustable drawoff device is also acceptable. The outlet structure should be designed so that the liquid level of the pond can be varied from a 3.0- 5.0 foot depth in increments of 0.5 foot or less. Withdrawal points shall be spaced so that effluent can be withdrawn from depths of 0.75 foot to 2.0 feet below pond water surface, irrespective of the pond depth.

The lowest drawoff lines should be 12 inches off the bottom to control eroding velocities and avoid pickup of bottom deposits. The overflow from the pond shall be taken near but below the water surface. A two-foot deep baffle may be helpful to keep algae from the effluent. The structure should also have provisions for draining the pond. A locking device should be provided to prevent unauthorized access to level control facilities. An unvalved overflow placed 6 inches above the maximum water level shall be provided.

Outlets should be located nearest the prevailing winds to allow floating solids to be blown away from effluent weirs.

The pond overflow pipes shall be sized for the peak design flow to prevent overtopping of the dikes.

9.3.2.4 Minimum and Maximum Pond Size

No pond should be constructed with less than 1/2 acre or more than 40 acres of surface area.

9.3.2.5 Number of Ponds

<u>A minimum of three ponds</u>, and preferably four ponds, <u>in series</u> should be provided (or baffling provided for a single cell lagoon design configuration) to insure good hydraulic design. The objective in the design is to eliminate short circuiting.

9.3.2.6 Parallel/Series Operation

Designs, other than single ponds with baffling, should provide for operation of ponds in parallel or series. Hydraulic design should allow for equal distribution of flows to all ponds in either mode of operation.

9.3.3 Aerated Lagoons

9.3.3.1 Depth

Depth should be based on the type of aeration equipment used, heat loss considerations, and cost, but should be no less than 7 feet. In choosing a depth, aerator erosion protection and allowances for ice cover and solids accumulation should be considered.

9.3.3.2 Influent Structures and Pipelines

The same requirements apply as described for facultative systems, except that the discharge locations should be coordinated with the aeration equipment design.

9.3.3.3 Interconnecting Piping and Outlet Structures

a. Interconnecting Piping

The same requirements apply as described for facultative systems.

b. Outlet Structure

The same requirements apply as described for facultative systems, except for variable depth requirements and arrangement of the outlet to withdraw effluent from a point at or near the surface. The outlet shall be preceded by an underflow baffle.

9.3.3.4 Number of Ponds

Not less than three basins should be used to provide the detention time and volume required. The basins should be arranged for both parallel and series operation. A settling pond with a hydraulic detention time of 2 days at average design flow must follow the
aerated cells, or an equivalent of the final aerated cell must be free of turbulence to allow settling of suspended solids.

9.3.3.5 Aeration Equipment

A minimum of two mechanical aerators or blowers shall be used to provide the horsepower required. At least three anchor points should be provided for each aerator. Access to aerators should be provided for routine maintenance which does not affect mixing in the lagoon. Timers will be required.

9.4 Pond Construction Details

9.4.1 Liners

9.4.1.1 Requirement for Lining

The seepage rate through the lagoon bottom and dikes shall not be greater than a water surface drop of 1/4 inch per day. (Note: The seepage rate of 1/4 inch per day is 7.3×10^{-6} cm/sec coefficient of permeability seepage rate under pond conditions.) If the native soil cannot be compacted or modified to meet this requirement, a pond liner system will be required.

If a lagoon is proposed to be upgraded, it must be shown that it currently meets the 1/4-inch per day seepage rate before approval will be given.

9.4.1.2 General

Pond liner systems that should be evaluated and considered include (1) earth liners, including native soil or local soils mixed with commercially prepared bentonite or comparable chemical sealing compound, and (2) synthetic membrane liners. The liner should not be subject to deterioration in the presence of the wastewater. The geotechnical recommendations should be carefully considered during pond liner design.
Consideration should also be given to construct test wells when required by the Department in any future regulations, or when industrial waste is involved.

9.4.1.3 Soil Liners

The thickness and the permeability of the soil liners shall be sufficient to limit the leakage to the maximum allowable rate of 1/4 inch per day. The evaluation of earth for use as a soil liner should include laboratory permeability tests of the material and laboratory compaction tests. The analysis should take into consideration the expected permeability of the soil when compacted in the field. All of the soil liner material shall have essentially the same properties.

The analysis of an earth liner should also include evaluation of the earth liner material with regard to filter design criteria. This is required so that the fine-grained liner material does not infiltrate into a coarser subgrade material and thus reduce the effective thickness of the liner.

If the ponds are going to remain empty for any period of time, consideration should be given to the possible effects on the soil liners from freezing and thawing during cold weather or cracking from hot, dry weather. Freezing and thawing will generally loosen the soil for some depth. This depth is dependent on the depth of frost penetration.

The compaction requirements for the liner should produce a density equal to or greater than the density at which the permeability tests were made. The minimum liner thickness should be 12 inches, to ensure proper mixing of bentonite with the native soil. The soil should be placed in lifts no more than 6 inches in compacted thickness. The moisture content at which the soil is placed should be at or slightly above the optimum moisture content.

Construction and placement of the soil liner should be inspected by a qualified inspector. The inspector should keep records on the uniformity of the earth liner material, moisture contents, and the densities obtained.

Bentonite and other similar liners should be considered as a form of earth liner. Their seepage characteristics should be analyzed as previously mentioned, and laboratory testing should be performed using the mixture of the native or local soil and bentonite or similar compound. In general, the requirements for bentonite or similar compounds should include the following: (1) The bentonite or similar compound should be high swelling and free flowing and have a particle size distribution favorable for uniform application and minimizing of wind drift; (2) the application rate should be least 125 percent of the minimum rate found to be adequate in laboratory tests; (3) application rates recommended by a supplier should be confirmed by an independent laboratory; and (4) the mixtures of soil and bentonite or similar compound should be compacted at a water content greater than the optimum moisture content.

9.4.1.4 Synthetic Membrane Liners

Requirements for the thickness of synthetic liners may vary due to the liner material, but it is generally recommended that the liner thickness be no less than 20 mils; that is, 0.020 inch. There may be special conditions when reinforced membranes should be considered. These are usually considered where extra tensile strength is required. The membrane liner material should be compatible with the wastewater in the ponds such that no damage results to the liner. PVC liners should not be used where they will be exposed directly to sunlight. The preparation of the subgrade for a membrane liner is important. The subgrade should be graded and compacted so that there are no holes or exposed angular rocks or pieces of wood or debris. If the subgrade is very gravelly and contains angular rocks that could possibly damage the liner, a minimum bedding of 3 inches of sand should be provided directly beneath the liner. The liner should be covered with 12 inches of soil. This includes the side slope as well. No equipment should be allowed to operate directly on the liner. Consideration should be given to specifying that the manufacturer's representative be on the job supervising the installation during all aspects of the liner placement. An inspector should be on the job to monitor and inspect the installation.

Leakage must not exceed 1/4-inch per day.

9.4.1.5 Other Liners

Other liners that have been successfully used are soil cement, gunite, and asphalt concrete. The performance of these liners is highly dependent on the experience and skill of the designer. Close review of the design of these types of liners is recommended.

9.4.2 Pond Construction

9.4.2.1 General

Ponds are often constructed of either a built-up dike or embankment section constructed on the existing grade, or they are constructed using a cut and fill technique. Dikes and embankments shall be designed using the generally accepted procedures for the design of small earth dams. The design should attempt to make use of locally available materials for the construction of dikes. Consideration should also be given to slope stability and seepage through and beneath the embankment and along pipes.

9.4.2.2 Top Width

The minimum recommended dike top width should be 12 feet on tangents and 15 feet on curves to permit access of maintenance vehicles. The minimum inside radius of curves of the corners of the pond should be 35 feet.

9.4.2.3 Side Slopes

Normally, inside slopes of either dikes or cut sections should not be steeper than 3 horizontal to 1 vertical. Outer slopes should not be steeper than 2 horizontal to 1 vertical. However, in many instances, the types of material used, maintenance considerations, and seepage conditions can indicate that other slopes should be used.

9.4.2.4 Freeboard

There should be sufficient freeboard to prevent overtopping of the dike from wave action and strong winds. A minimum of one foot is required.

9.4.2.5 Erosion Control

Erosion control should be considered for the inside slopes of the dike to prevent the formation of wavecut beaches in the dike slope. In the event that earth liners or membrane liners with earth cover are used, consideration should be given to erosion protection directly beneath aeration units. If the currents are strong enough, considering the type of material used for the earth cover, erosion pads may be necessary beneath the aeration units. Erosion control should also be considered wherever influent pipes empty into the pond. If a grass cover for the outer slopes is desired, they should be fertilized and seeded to establish a good growth of vegetative cover. This vegetative cover will help control erosion from runoff. Consideration should also be given to protection of the outer slopes in the event that flooding occurs. The erosion protection should be able to withstand the currents from a flood.

9.4.3 Prefilling

The need to prefill ponds in order to determine the leakage rate shall be determined by the Department and incorporated into the plans and specifications. The strongest consideration for prefilling ponds will be given to ponds with earth liners. Ponds in areas where the surrounding homes are on wells will also be given strong consideration for prefilling.

9.4.4 Utilities and Structures Within Dike Sections

Pipes that extend through an embankment should be bedded up to the springline with concrete. Backfill should be with relatively impermeable material. No granular bedding material should be used. Cutoff collars should be used as required. No gravel or granular base should be used under or around any structures placed in the embankment within the pond. Embankments should be constructed at least 2 feet above the top of the pipe before excavating the pipe trench.

9.5 Hydrograph Controlled Release (HCR) Lagoons

All lagoons requirements apply to HCR lagoons with the following additional concerns:

HCR lagoons control the discharge of treated wastewater in accordance with the stream's assimilative capacity. Detention times vary widely and must be determined on a case-by-case basis.

HCR sites require much receiving stream flow pattern characterization. For this purpose, EPA Region IV has developed a computer design program. The Division of Water Pollution Control can assist in sizing the HCR basin using this program. HCR sites may be more economical if the design is combined with summertime land application. Their design is more economical if summer/winter or monthly standards are available.

The design and construction of the in-stream flow measurement equipment are critical components of an HCR system. The United States Geological Survey (USGS) should be contacted during the design phase. The USGS also has considerable construction experience concerning in-stream monitoring stations, although construction need not necessarily be done or supervised by the USGS.

9.6 Polishing Lagoons

Polishing lagoons following activated sludge are not permissible in Tennessee due to the one-cell algae interference.

9.7 Operability

Once a pond is designed, little operation should be required. However, to avoid NPDES permit violations, pond flexibility is needed. Operation flexibility is best facilitated by the addition of piping and valves to each pond which allows isolation of its volume during an algal bloom.

9.8 Upgrading Existing Systems

There are approximately sixty existing lagoons in Tennessee which were built utilizing standards and criteria from the 1960 period. Most are single- or double-cell units which need upgrading. Many are required to meet tertiary standards. The upgrade case should, in general, utilize the guidance in this chapter or proven configurations. It is noted, however, that there are many lagoon combinations available, such as complete-mix pond, partial-mix pond, stabilization pond, HCR pond and marsh-pond (wetlands)concepts. The combination of these alternatives
should be based upon the effluent permit design standards as well as site economics.

4 AUGUST 1995

CHAPTER 10

Disinfection

DISINFECTION

10.1 General

10.1.1 Requirement for Disinfection

Proper disinfection of treated wastewater before disposal is required for all plants (with the exception of some land application systems) to protect the public health.

Disinfection as a minimum shall:

a. Protect public water supplies

b. Protect fisheries and shellfish waters

c. Protect irrigation and agricultural waters

d. Protect water where human contact is likely

10.1.2 Methods of Disinfection

10.1.2.1 Chlorination

Chlorination using dry chlorine (see definition in following section) is the most commonly applied method of disinfection and should be used unless other factors, including chlorine availability, costs, or environmental concerns, justify an alternative method.

10.1.2.2 Ozonation

Ozonation may be considered as an alternative to chlorination for the reasons described above. Ozonation is considered as Developmental Technology, and should only be considered for very large installations.

10.1.2.3 Other

Other potential methods of disinfection, such as by ultraviolet light, are available and their application will be considered on a case-by-case basis.

10.1.3 Dechlorination

Capability to add dechlorination should be considered in all new treatment plants. Dechlorination of chlorinated effluents shall be provided when permit conditions dictate the need.

10.2 Chlorination

10.2.1 General

10.2.1.1 Forms of Chlorine

a. Dry Chlorine

Dry chlorine is defined as elemental chlorine existing in the liquid or gaseous phase, containing less than 150 mg/l water. Unless otherwise stated, the word "chlorine" wherever used in this section refers to dry chlorine.

b. Sodium Hypochlorite

Sodium hypochlorite may be used as an alternative to chlorine whenever dry chlorine availability, cost, or public safety justifies its use. The requirements for sodium hypochlorite generation and feeding will be determined on a case-by-case basis.

c. Other

Other chlorine compounds such as chlorine dioxide or bromine chloride may be used as alternatives to chlorine whenever cost or environmental concerns justify their use. The acceptability of other chlorine compounds will be determined on a case-by-case basis.

10.2.1.2 Chlorine Feed Equipment

Solution-feed vacuum-type chlorinators are generally preferred for large installations. The use of hypochlorite feeders of the positive displacement type may be considered. Dry chlorine tablet type feeders may also be considered for small flows, into large streams.

Liquid chlorine evaporators should be considered where more than four 1-ton containers will be connected to a supply manifold.

10.2.1.3 Chlorine Supply

a. Cylinders

Cylinders should be considered where the average daily chlorine use is 150 pounds or less. Cylinders are available in 100-pound or 150-pound sizes.

b. Containers

The use of 1-ton containers should be considered where the average daily chlorine consumption is over 150 pounds.

c. Large-Volume Shipments

At large installations, consideration should be given to the use of truck or railroad tank cars, or possibly barge tank loads, generally accompanied by gas evaporators.

10.2.1.4 Chlorine Gas Withdrawal Rates

The maximum withdrawal rate for 100- and 150- pound cylinders should be limited to 40 pounds per day per cylinder.

When gas is withdrawn from 2,000-pound containers, the withdrawal rate should be limited to 400 pounds per day per container.

10.2.2 Design Considerations

10.2.2.1 General

Chlorination system designs should consider the following design factors:

Flow

Contact time

Concentration and type of chlorine residual

Mixing

pH

Suspended solids

Industrial wastes

Temperature

Concentration of organisms

Ammonia concentration

10.2.2.2 Capacity

Required chlorinator capacities will vary, depending on the use and point of application of the chlorine. Chlorine dosage should be established for each individual situation, with those variables affecting the chlorine reaction taken into consideration. For normal wastewater, the following dosing capacity may be used as a guideline.

Type of Treatment	Dosage Capacity* (mg/l)
Prechlorination for Odor Control	20-25
Activated Sludge Return	5-10
Trickling Filter Plant Effluent (non-nitrified)	3-15
Activated Sludge Plant Effluent (non-nitrified)	2-8
Tertiary Filtration Effluent	1-6
Nitrified Effluent	2-6

Stabilization Pond Effluent	Up to 35

* Based on Average Design Flow.

The design should provide adequate flexibility in the chlorination equipment and control system to allow controlled chlorination at minimum and peak flows over the entire life of the treatment plant. Special consideration should be given to the chlorination requirements during the first years of operation to ensure the chlorination system is readily operable at less than design flows without overchlorination. Chlorination equipment should operate between 25% and 75% of total operating range, to allow for adjusting flexibility at design average flow.

10.2.2.3 Mixing

The mixing of chlorine and wastewater can be accomplished by hydraulic or mechanical mixing.

Hydraulic mixing is preferred in smaller plants over mechanical mixing and should be done according to the following criteria.

a. Pipe Flow:

A Reynolds Number of greater than or equal to 1.9×10^4 is required.

Pipes up to 30 inches in diameter: chlorine injected into center of pipe.

Pipes greater than 30 inches in diameter: chlorine injected with a grid-type diffuser.

Chlorine applied at least 10 pipe diameters upstream from inlet to contact tank.

b. Open channel flow: a hydraulic jump with a minimum Froude Number of 4.5 is necessary to provide adequate hydraulic mixing. Point of chlorine injection must be variable because jump location will change with changes in flow.

When mechanical mixing must be used, the following criteria apply:

Use where Reynolds Number for pipe flow is less than 1.9×10^4 or for open channel flow without a hydraulic jump.

A mixer-reactor unit is necessary that provides 6 to 18 seconds contact.

Inject chlorine just upstream from mixer.

Mixer speed a minimum of 50 revolutions per minute (rpm).

Jet Chlorinators may be used in a separate chamber from the contact chamber. The contact chamber shall conform to Section 10.2.2.4 with an average design flow minimum detention time reduced to 15 minutes and a peak detention time of 7.5 minutes.

10.2.2.4 Contact Period

Contact chambers shall be sized to provide a minimum of 30 minutes detention at average design flow and 15 minutes detention at daily peak design flow, whichever is greater. Contact chambers should be designed so detention times are less than 2 hours for initial flows.

10.2.2.5 Contact Chambers

The contact chambers should be baffled to minimize short-circuiting and backmixing of the chlorinated wastewater to such an extent that plug flow is approached. It is recommended that baffles be constructed parallel to the longitudinal axis of the chamber with a minimum length-to-width ratio of 30:1 (the total length of the channel created by the baffles should be 30 times the distance between the baffles). Shallow unidirectional contact chambers should also have cross-baffles to reduce short-circuiting caused by wind currents.

Provision shall be made for removal of floating and settleable solids from chlorine contact tanks or basins without discharging inadequately disinfected effluent. To accomplish continuous disinfection, the chlorine contact tank should be designed with duplicate compartments to permit draining and cleaning of individual compartments. A sump or drain within each compartment, with the drainage flowing to a raw sewage inlet, shall be provided for dewatering, sludge accumulation, and maintenance. Unit drains shall not discharge into the outfall pipeline. Baffles shall be provided to prevent the discharge of floating material.

A readily accessible sampling point shall be provided at the outlet end of the contact chamber.

In some instances, the effluent line may be used as chlorine contact chambers provided that the conditions set forth above are met.

10.2.2.6 Dechlorination

a. Sulfur Dioxide

Sulfur dioxide can be purchased, handled, and applied to wastewater in the same way as chlorine. Sulfur dioxide gas forms sulfurous acid, a strong reducing agent, when combined with water. When mixed with free and combined chlorine residuals, sulfurous acid will neutralize these active chlorine compounds to the nontoxic chloride ion. Sulfur dioxide dosage required for dechlorination is 1 mg/l of SO_2 for 1 mg/l of chlorine residual expressed as Cl_2. Reaction time is essentially instantaneous. Detention time requirements are based on the time necessary to assure complete mixing of the sulfur dioxide.

b. Other Methods

For very small treatment systems, detention ponds should be considered for dechlorination.

Design rationale and calculations shall be submitted upon request to justify the basis of design for all major components of other dechlorination processes.

10.2.2.7 Sampling, Instrumentation, and Control

For treatment facility designs of 0.5 mgd and greater, continuously modulated dosage control systems should be used. The control system should adjust the chlorine dosage rate to accommodate fluctuations in effluent chlorine demand and residual caused by changes in waste flow and waste characteristics with a maximum lag time of five minutes. These facilities should also utilize continuous chlorine residual monitoring.

Flow proportional control is preferred over manual control for smaller facilities and may be required on a case-by-case basis. The design shall shut off the chlorination for small systems where the flow is zero, such as late at night.

In all cases where dechlorination is required, a compound loop control system or equivalent should be provided.

All sample lines should be designed so that they can be easily purged of slimes and other debris and drain or be protected from freezing.

Alarms and monitoring equipment that adequately alert the operators in the event of deficiencies, malfunctions, or hazardous situations related to chlorine supply metering equipment, leaks, and residuals may be required on a case-by-case basis.

Design of instrumentation and control equipment should allow operation at initial and design flows.

10.2.2.8 Residual Chlorine Testing

Equipment should be provided for measuring chlorine residual. There are five EPA accepted methods for analysis of total residual chlorine and they are 1) Ion Selective Electrode, 2) Amperometric End Point Titration Method, 3) Iodometric Titration Methods I & II, 4) DPD Colormetric Method and, 5) DPD Ferrous Titrimetric Method. Where the discharge occurs in critical areas, the installation of facilities for continuous automatic chlorine residual analysis and recording systems may be required.

10.2.3 Design Details

10.2.3.1 Housing

a. General

An enclosed structure shall be provided for the chlorination equipment.

Chlorine cylinder or container storage area shall be shaded from direct sunlight.

Chlorination systems should be protected from fire hazards, and water should be available for cooling cylinders or containers in case of fire.

Any building which will house chlorine equipment or containers should be designed and constructed to protect all elements of the chlorine system from fire hazards. If flammable materials are stored or processed in the same building with chlorination equipment (other than that utilizing hypochlorite solutions), a firewall should be erected to separate the two areas.

If gas chlorination equipment and chlorine cylinders or containers are to be in a building used for other purposes, a gastight partition shall separate this room from any other portion of the building. Doors to this room shall open only to the outside of the building and shall be equipped with panic hardware. Such rooms should be at or above ground level and should permit easy access to all equipment.

A reinforced glass, gastight window shall be installed in an exterior door or interior wall of the chlorinator room to permit the chlorinator to be viewed without entering the room.

Adequate room must be provided for easy access to all equipment for maintenance and repair. The minimum acceptable clearance around and in back of equipment is 2 feet, except for units designed for wall or cylinder mounting.

b. Heat

Chlorinator rooms should have a means of heating and controlling the room air temperature above a minimum of 55° F. A temperature of 65° F is recommended.

The room housing chlorine cylinders or containers in use should be maintained at a temperature less than the chlorinator room, but in no case less than 55° F unless evaporators are used and liquid chlorine is withdrawn.

All rooms containing chlorine should also be protected from excess heat.

The room containing ozone generation units shall be maintained above 35°F at all times.

c. Ventilation

All chlorine feed rooms and rooms where chlorine is stored should be force-ventilated, providing one air change per minute, except "package" buildings with less than 16 square feet of floor space, where an entire side opens as a door and sufficient cross-ventilation is provided by a window. For ozonation systems, continuous ventilation to provide at least 6 complete air changes per hour should be installed. The entrance to the air exhaust duct from the room should be near the floor and the point of discharge should be so located as not to contaminate the air inlet to any building or inhabited areas. The air inlet should be located to provide cross-ventilation by air at a temperature that will not adversely affect the chlorination equipment.

Chlorinators and some accessories require individual vents to a safe outside area. The vent should terminate not more than 25 feet above the chlorinator or accessory and have a slight downward slope from the highest point. The outside end of the vent should bend down to preclude water entering the vent and be covered with a screen to exclude insects.

d. Electrical

Electrical controls for lights and the ventilation system should operate automatically when the entrance doors are opened. Manually controlled override switches should be located adjacent to and outside of all entrance doors, with an indicator light at each entrance. Electrical controls should be excluded, insofar as possible, from rooms containing chlorine cylinders, chlorine piping, or chlorination equipment.

e. Dechlorination equipment (SO_2) shall not be placed in the same room as the Cl_2 equipment. SO_2 equipment is to be located such that the safety requirements of handling Cl_2 are not violated in any form or manner.

10.2.3.2 Piping and Connections

a. Dry Chlorine

Piping systems should be as simple as possible, with a minimum number of joints; piping should be well supported, adequately sloped to allow drainage, protected from mechanical damage, and protected against temperature extremes.

The piping system to handle gas under pressure should be constructed of Schedule 80 black seamless steel pipe with 2,000-pound forged steel fittings. Unions should be ammonia type with lead gaskets. All valves should be Chlorine Institute-approved. Gauges should be equipped with a silver protector diaphragm.

Piping can be assembled by either welded or threaded connections. All threaded pipe must be cleaned with solvent, preferably trichlorethylene, and dried with nitrogen gas or dry air. Teflon tape should be used for thread lubricant in lieu of pipe dope.

b. Injector Vacuum Line

The injector vacuum line between the chlorinator and the injector should be Schedule 80 PVC or fiber cast pipe approved for moist chlorine use.

c. Chlorine Solution

The chlorine solution lines can be Schedule 40 or 80 PVC, rubber-lined steel, saran-lined steel, or fiber cast pipe approved for moist chlorine use. Valves should be PVC, PVC-lined, or rubber- lined.

10.2.3.3 Water Supply

An ample supply of water shall be available for operating the chlorinator. Where a booster pump is required, duplicate equipment shall be provided, and, when necessary, standby power as well. When connection is made from domestic water supplies, equipment for backflow prevention shall be provided. Where treated effluent is used, a wye strainer shall be required. Pressure gauges should be provided on chlorinator water supply lines.

10.2.3.4 Standby Equipment and Spare Parts

Standby chlorination capabilities should be provided which will ensure adequate disinfection with any unit out of operation for maintenance or repairs. An adequate inventory of parts subject to wear and breakage should be maintained at all times.

10.2.3.5 Scales

Scales shall be provided at all plants using chlorine gas. At large plants, scales of the indicating and recording type are recommended. Scales shall be provided for each cylinder or container in service; one scale is adequate for a group of cylinders or containers connected to a common manifold. Scales should be constructed of or coated with corrosion-resistant material.

Scales shall be recommended for day tanks when using HTH.

10.2.3.6 Handling Equipment

Handling equipment should be provided as follows for 100- and 150-pound cylinders:

A hand truck specifically designed for cylinders

A method of securing cylinders to prevent them from falling over

Handling equipment should be provided as follows for 2,000-pound containers:

Two-ton-capacity hoist

Cylinder lifting bar

Monorail or hoist with sufficient lifting height to pass one cylinder over another

Cylinder trunnions to allow rotating the cylinders for proper connection.

10.2.3.7 Container Space

Sufficient space should be provided in the supply area for at least one spare cylinder or container for each one in service.

10.2.3.8 Automatic Switchover of Cylinders and Containers

Automatic switchover of chlorine cylinders and containers at facilities having less than continuous operator attendance is desirable and will be required on a case-by-case basis.

10.2.4 Safety

10.2.4.1 Leak Detection and Controls

A bottle of 56% ammonium hydroxide solution shall be available for detecting chlorine leaks.

All installations utilizing 2,000-pound containers and having less than continuous operator attendance shall have suitable continuous chlorine leak detectors. Continuous chlorine leak detectors would be desirable at all installations. Whenever chlorine leak detectors are installed, they should be connected to a centrally located alarm system and shall automatically start exhaust fans.

10.2.4.2 Breathing Apparatus

At least one gas mask in good operating condition and of a type approved by the National Institute for Occupational Safety and Health (NIOSH) as suitable for high concentrations of chlorine gas shall be available at all installations where chlorine gas is handled and shall be stored outside of any room where chlorine is used or stored. Instructions for using, testing, and replacing mask parts, including canisters, shall be posted. At large installations, where 1-ton containers are used, self- contained air breathing apparatus of the <u>positive pressure type</u> shall be provided.

10.2.4.3 Container Repair Kits

All installations utilizing 1-ton containers should have Chlorine Institute Emergency Container Kits. Other installations using cylinders should have access to kits stored at a central location.

10.2.4.4 Piping Color Codes

It is desirable to color code all piping related to chlorine systems.

10.3 <u>Alternate Methods</u>

10.3.1 Ozonation

10.3.1.1 Application

Ozonation may be substituted for chlorination whenever chlorine availability, cost, or environmental benefits justify its application.

Ozone is generated on-site from either air or high-purity oxygen. Ozonation should be considered if high-purity oxygen is available at the plant for other processes.

10.3.1.2 Design Basis

The design requirements for ozonation systems should be based on pilot testing or similar full-scale installations. As a minimum, the following design factors should be considered:

a. Ozone dosage

b. Dispersion and mixing of ozone in wastewater

c. Contactor design

All design criteria shall be submitted upon request to justify the basis of design of the ozonation system. The detailed design requirements will be determined on a case-by-case basis.

10.3.2 Ultraviolet Disinfection

10.3.2.1 Application

UV disinfection may be substituted for chlorination, particularly whenever chlorine availability, cost, or environmental benefits justify its application. For tertiary treatment plants where dechlorination is required or chlorine toxicity is suspected, UV disinfection is a viable alternative.

10.3.2.2 Design Basis

In the design of UV disinfection units there are three basic areas that should be considered:

a. Reactor hydraulics

b. Factors affecting transmission of UV light to the microorganisms

c. Properties of the wastewater being disinfected.

UV disinfection is considered as Developmental Technology and all design criteria shall be submitted upon request to justify the basis of the UV disinfection system. The detailed design requirements will be determined on a case-by-case basis.

CHAPTER 11

Tertiary Treatment/Advanced Wastewater Treatment

11.1 Filtration

11.1.1 General
11.1.2 High Rate Gravity Filters
11.1.3 Pressure and Vacuum High Rate Filters
11.1.4 Standard Rate Gravity Filters
11.1.5 Shallow Bed Filters (Slow Sand Filters)
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11.2 Post Aeration

11.2.1 General
11.2.2 Aeration Tank Systems
11.2.3 Cascade Systems
11.2.4 Operability

11.3 Nutrient Removal

11.1 Filtration

11.1.1 General

Supplementary solids separation, following secondary clarification of wastewater, may be needed either as a final treatment step or prior to discharging to an ion exchange bed, carbon bed, reverse osmosis or other system. Filtration should be accomplished through a filter consisting of sand; sand and anthracite; anthracite; or anthracite, sand and garnet (or ilmenite).

11.1.2 High Rate Gravity Filters

11.1.2.1 Design

A minimum wastewater depth of 3 feet, measured from the normal operating wastewater surface to the surface of the filter medium, shall be provided. Even distribution of the wastewater over the filter area shall be provided. The top filter material shall not be displaced by the influent wastewater. The bottom washwater trough elevation shall be above the maximum level of expanded medium during backwashing. A top washwater trough elevation shall be no more than 30 inches above the filter surface. Spacing of the troughs shall be such that horizontal partical travel distance is not greater than 3 feet, and equal spacing between troughs is provided so that the same number of square feet of filter area is served by each trough.

For High Rate Filtration, dual or multi-media only shall be used. The maximum filter rate shall be 4 gpm/ft^2 immediately after backwash with a nominal rate of less than 4 gpm/ft^2 at the peak daily flow. A minimum of two filters shall be provided. Filtration shall be designed so that, with one filter out of service, each of the remaining filter(s) shall filter no greater than 4 gpm/ft^2 at the design peak daily flow. Equipment for the application of filter aids to the filter influent should be provided.

11.1.2.2 Medium

a. Sand - The medium shall be clean silica sand having

   (i) a depth of 30 inches;

   (ii) an effective size of from 0.35 mm to 0.55 mm, depending upon the loading of the wastewater, and;

   (iii) a uniformity coefficient not greater than 1.70.

b. Anthracite - a combination of sand and clean crushed anthracite may be used. The anthracite shall have

   (i) an effective size of 0.8 mm - 1.2 mm, and;

   (ii) a uniformity coefficient not greater than 1.85;

   (iii) anthracite layer shall not exceed 20 inches in a 30-inch bed.

c. A 3-inch layer of torpedo sand may be used as a supporting medium for the filter sand; such torpedo sand shall have

(i) an effective size of 0.8 mm to 2.0 mm, and,

(ii) a uniformity coefficient not greater than 1.70.

d. Gravel - Gravel, when used as the supporting medium, shall consist of hard, rounded silicious particles.

(i) The minimum gravel size of the bottom layer should be 3/4 inch or larger.

(ii) For proper grading of intermediate layers:

(1) the minimum particle size of any layer should be as large as the maximum particle size in the layer next above and;

(2) within any layer the maximum particle size should not be more than twice the minimum particle size.

(iii) The depth of any gravel layer should not be less than 2 inches or less than twice the largest gravel size for that layer, whichever is greater. The bottom layer should be thick enough to cover underdrain laterals, strainers, or other irregularities in the filter bottom.

(iv) The total depth of gravel above the underdrains should not be less than 10 inches.

(Reduction of gravel depths may be considered upon justification when proprietary filter bottoms are installed.)

e. Multi-media - To be approved on a case-by-case basis.

The medium should consist of anthracite, silica sand, and/or other suitable sand. Since filters presently utilizing dual media and mixed media are proprietary in nature, no attempt will be made to set standards for minimum filter media depth, effective size and uniformity coefficient of filter media, or the specific gravity of that medium.

11.1.2.3 Underdrains.

Porous-plate bottoms shall not be used. Perforated pipe underdrains should be used, consisting of a manifold and laterals. Underdrain systems allowable in water plants such as Leopold or Wheeler bottoms are acceptable. The orifice loss in backwashing must exceed the sum of the minor hydraulic losses in the underdrain system to secure good distribution of flow over the entire area of the filter bottom. In order to insure adequate design of perforated pipe underdrain systems the following ratios must fall within the ranges shown:

$$\frac{\text{orifice area}}{\text{bed area}} = \frac{0.0015}{1} \text{ to } \frac{0.005}{1}$$

$$\frac{\text{lateral area}}{\text{area of orifices served}} = \frac{2}{1} \text{ to } \frac{4}{1}$$

$$\frac{\text{manifold area}}{\text{area of laterals served}} = \frac{1.5}{1} \text{ to } \frac{3}{1}$$

Orifices should have 3 to 12 inch spacing, and laterals the same. Underdrains should be made of corrosion and scale resistant materials, or properly protected against corrosion.

Orifices through false filter bottoms or underdrain design are preferred. The glazed tile filter block used in some filter bottoms and the stainless steel modulars used in other filter bottom designs are recommended to provide even and uniform distribution of backwash water. Hydraulic distribution data on each standard filter size should be submitted.

11.1.2.4 Backwash

Provisions shall be made for washing filters as follows:

a. a rate to provide for a 50 percent expansion of the medium is recommended, consistent with water temperatures and specific gravity of the filter medium; a minimum rate of 15 gpm/ft^2 is recommended, however 20 gpm/ft^2 may be required for adequate expansion of the filter medium.

b. filtered wastewater provided at the required backwash rate by washwater tanks, a washwater pump(s) or a combination of these is required,

c. washwater pumps in duplicate unless an alternate means of obtaining washwater is available; air release must be provided;

d. washwater supply to backwash two filters for at least 5 minutes at the design rate of wash; plus surface wash requirements;

e. A washwater regulator or valve on the main washwater line to obtain the desired rate of filter wash with the washwater valves on the individual filters completely open is required.

f. Air scouring at 3-5 cu ft/min/ft^2 of filter area for at least 3 minutes preceding water backwash is acceptable.

g. Rate of flow indicators on the main washwater line shall be provided and should be located so that it can be easily read by the operator during backwash.

h. Backwash wastewater treatment and disposal must be accomplished within the rated design capacities of the treatment system. Backwash wastewater cannot be discharged to a stream without first receiving adequate treatment. If it is desired to recycle the backwash wastewater through a secondary system, then the hydraulic design of the entire system (including the clarifier and filter) must be based on the anticipated rate of raw influent flow plus the flow rate at which the backwash water enters the system. In most systems a backwash water holding tank and controlled discharge system will be required. This holding system must be capable of storing the wastes from two backwashes and discharging the wastes to the treatment system within 24 hours at a rate which, in combination with the raw influent, does not exceed the hydraulic design of any system component when the loading period for the plant is 24 hours. For plants with loading periods less than 24 hours, additional backwash holding capacity may be required. For example, a school's sewage treatment plant with an 8-hour loading period and a backwash holding system which pumps from its holding tank to the head of the treatment process only during low loading periods may require a holding tank with a capacity for three or more backwash volumes.

i. Backwash may be initiated either automatically or manually; the length of the backwash period must be automatically controlled by a timing device adjustable in one minute increments up to a possible 15 minute backwash duration.

11.1.2.5 Surface Wash

Surface wash facilities are required. Disinfected filtered wastewater effluent should be used for surface wash. Revolving-type surface washers should be provided; however, other types may be considered. All rotary surface wash devices should be designed with:

a. Provisions for minimum washwater pressures of 40 psi and;

b. Provisions for adequate surface washwater to provide 0.5 to 1 gallon per minute per square foot of filter area.

11.1.3 Pressure and Vacuum High Rate Filters

11.1.3.1 General

Pressure sand filters are those operating under pressure in a closed container. Generally, a pump discharge line delivers the influent to the pressure filter. Vacuum sand filters are those operating under partial vacuum within the underdrain system; they can have open beds. Generally, a pump suction line is connected to the underdrain of a vacuum sand filter.

11.1.3.2 Design

Design requirements for pressure or vacuum filters include all of those listed for High Rate Gravity Filters in paragraphs 11.1.2.1 through 11.1.2.5, plus the following;

Pressure filter containers must meet all applicable safety codes and requirements. Containers must be large enough to permit a man to work inside for medium removal and underdrain maintenance. A minimum diameter of 3 feet is suggested. An access port must be provided for inspection and maintenance purposes.

11.1.4 Standard Rate Gravity Filters

11.1.4.1 General

A minimum of two complete units is required. Each unit must be designed to treat 100 percent of plant flow except where design flow is 100,000 gpd or greater (see Design Section 11.1.4.2). The sand surface must be submerged at all times. Generally, standard rate filters are monomedium sand filters (see Media Section 11.1.4.3).

11.1.4.2 Design

The hydraulic design loading for each filter must be within the range of 1.0 to 2.0 gpm/ft^2. For installation less than 100,000 gpd the nominal filter rate shall be 1.0 gpm/ft^2 with one cell loaded no more than 2.0 gpm/ft^2 during backwash of the other cell. For installations greater than 100,000 gpd it is expected that each filter cell will be loaded at 2 gpm/ft^2 and during periods of backwash; no other cell may be loaded higher than 4 gpm/ft^2. Even distribution of the wastewater over the filter shall be provided. The filter sand shall not be displaced by the influent wastewater. The bottom washwater trough elevation shall be above the maximum level of expanded medium during backwash. A top washwater trough elevation shall be no more than 30 inches above the filter surface. Spacing of the troughs shall be such that horizontal partical travel distance is not greater than 3 feet, and equal spacing between troughs is provided so that the same number of square feet of filter area is served by each trough.

11.1.4.3 Medium

The filter medium should have the following properties:

a. Sand

A sieve analysis should be provided by the design engineer. The medium should be clean silica sand having (1) a depth of not less than 27 inches and generally not more than 30 inches after cleaning and scraping and (2) an effective size of 0.35 mm to 0.5 mm, depending upon the quality of the applied wastewater, and a uniformity coefficient not greater than 1.6. Clean crushed anthracite or a combination of sand and anthracite may be used. Such media should have (1) an effective size from 0.45 mm to 0.8 mm and (2) a uniformity coefficient not greater than 1.7.

b. Supporting medium for the filter sand

A sieve analysis should be provided by the design engineer. A 3-inch layer of torpedo sand should be used as the supporting medium for the filter sand. Such torpedo sand should have (1) an effective size of 0.8 mm to 2.0 mm and (2) a uniformity coefficient not greater than 1.7.

c. Gravel

Gravel when used as a supporting medium should consist of hard, rounded particles and should not include flat or elongated particles. The coarsest gravel should be 2 1/2 inches in diameter when the gravel rests directly on the strainer system and should extend above the top of the perforated laterals or strainer nozzles. Not less than four layers of gravel should be used.

11.1.4.4 Underdrains

All requirements of Section 11.1.2.3 apply.

11.1.4.5 Backwash

All requirements of Section 11.1.2.4 apply with the additional consideration:

There shall be the capability to backwash at a rate of 20 gpm/ft^2 for adequate expansion of the filter medium.

11.1.4.6 Surface Wash

All requirements of Section 11.1.2.5 apply.

11.1.5 Shallow Bed Filters (Slow Sand Filters)

These filters are normally used at small treatment facilities and will be reviewed on a case-by-case basis.

11.1.6 Operability

11.1.6.1 The clear well must be protected to keep unfiltered effluent from entering the clear well in the event that some accident or malfunction causes a filter to overflow.

11.1.6.2 It is suggested that a supplementary clean water source, such as a high volume hydrant (protected by a back-flow prevention device) be available for filling the clear well.

11.1.6.3 Any wastewater treatment facility that has a flow peaking factor equal to or greater than 1.5 shall have an equalization/surge tank to control filtration rate. The size of the equalization/surge tank must be determined on the basis of rate and duration of peak flows including the recirculated backwash water. For systems with a flow peaking factor less than 1.5, the rate of filtration may be accomplished by valves in such a way that will not cause water to surge through the filter at rates higher than design. Position indicators must be provided for automatic valves. Pressure or head loss gages must be provided on the influent and effluent side of each filter. Micro switches will also be acceptable. On larger installations (75,000 gpd or greater) a rate of flow indicator will be required. Rapid variations of filtration rate are undesirable as they may cause dislodging of deposited matter and subsequent deterioration of effluent quality.

11.1.6.4 A by-pass around the filters must be provided and controlled by an easily accessible valve with markings for open or closed positions.

11.1.6.5 The capability to disinfect both prior to and after the filters shall be provided.

11.1.6.6 Vertical walls within the filter are required unless otherwise approved.

11.1.6.7 There shall be no protrusion of the filter walls into the filter medium.

11.1.6.8 Sufficient head room shall be provided when filters are indoors to permit normal inspection and operation.

11.1.6.9 The minimum depth of filter shall be 8 feet.

11.1.6.10 Trapped effluent to prevent backflow of air to the bottom of the filters is required.

11.1.6.11 Washwater drain capacity shall be designed to carry maximum flow.

11.1.6.12 Walkways around filters, not less than 24 inches wide, shall be provided where the installation is above ground level.

11.1.6.13 When backwash is automatically controlled, the backwash rate shall increase gradually or "step up" in a manner so to not displace the media or "blow" the filter bottom with a sudden surge.

11.2 Post Aeration

11.2.1 General

Post aeration is used to maintain a required minimum dissolved oxygen residual in treated wastewater effluent. Post aeration is often needed following a dechlorination process where an oxygen depleting chemical such as sulfur dioxide is used.

11.2.2 Aeration Tank Systems

Design consists of determining the oxygen requirements and providing sufficient oxygen transfer capability to satisfy these requirements. The design should consider the quantity of oxygen to satisfy the oxygen deficit required to meet the receiving water standards plus the oxygen-utilization rate of the effluent wastewater. Design of the oxygen transfer equipment in an aeration tank stage should be based on the final dissolved oxygen leaving that aeration tank stage. Design of aeration tanks and equipment should conform to the pertinent requirements of Chapter 7, "Activated Sludge."

Calculations shall be submitted to justify the basis of design.

Aeration equipment may be any of the following;

1. Fine-bubble diffused air
2. High or Low speed surface aerators
3. Submerged turbine
4. High-purity oxygen

Other types will be considered based on performance and design data submitted with the request.

11.2.3 Cascade Systems

Cascade aeration consists of a series of steps or weirs over which the wastewater is passed in thin layers to maximize turbulence and promote transfer of atmospheric oxygen. The engineer shall demonstrate that the design will meet the receiving water standards either by use of data from the literature or pilot testing. Calculations shall be submitted to justify the basis of design.

11.2.4 Operability

11.2.4.1 The design should incorporate provisions for the control of foam.

11.2.4.2 A series of basins may improve transfer efficiency and also reduce total horsepower required as opposed to one large basin.

11.2.4.3 Baffles should be used with mechanical aerators to prevent vortexing.

11.3 <u>Nutrient Removal</u>

______Nutrient removal, either supplementary or incorporated within standard secondary treatment facilities may be required in areas where receiving waters are greatly used and re-used or where highly restrictive use classifications have been established. For organization purposes, a very broad definition of "nutrients" shall be adopted herein to include refractory organics, nitrogen, phosphorus and inorganic salts. Sufficient operating data and information are not available to permit the establishment of detailed criteria outlining the proper application of the various available processes and operations to a specific treatment situation. Until sufficient operating data are obtained, the development and design of nutrient removal processes must be based upon the best obtainable pilot plant data (developed by the application of standard processes and operations to the specific waste treatment problem on a small scale basis). In order for approval of any type of supplementary nutrient removal system, sufficient pilot plant operating data must be made available to allow an evaluation of the adequacy and efficacy of the proposed process. No process will be approved unless adequate provisions are made for the ultimate disposal of concentrated pollutants "created" by the process (such as spent ion exchange regenerants, concentrated brines from reverse osmosis and electrodialysis systems, contaminated sorption media, chemical sludges and so forth).

CHAPTER 12

Sludge Processing and Disposal

SLUDGE PROCESSING AND DISPOSAL

12.1 General

12.1.1 Definition

Sludge is a broad term used to describe the various aqueous suspensions of solids encountered during treatment of sewage. The nature and concentration of the solids control the processing characteristics of the sludge. Grit screenings and scum are not normally considered as sludge and therefore are not discussed in this section.

12.1.2 Total Systems Approach to Design

The most frequently encountered problem in wastewater treatment plant design is the tendancy to optimize a given subsystem, such as sludge dewatering, without considering the side effects of this optimization on the overall plant operation and treatment costs.

Sludge handling processes can be classified as thickening, conditioning, stabilization, dewatering, and disposal. Numerous process alternatives exist within each of these categories. Each unit process should be evaluated as part of the total system, keeping in mind that the objective is to use that group of processes that provides the most cost-effective method of sludge disposal.

The analysis should include a materials balance to identify the amounts of material which enter, leave, accumulate, or are depleted in the given process and the system as a whole. Energy requirements should also be provided to aid in determining capital and operating costs of the total system.

12.1.3 Recycle Streams

Recycle streams from the process alternatives, including thickener overflow, centrate, filtrate, and supernatant, should be returned to the sewage treatment process at appropriate points to maintain effluent quality within the limits established. Volume and strength of each recycle stream should be considered in the plant design. Sidestream treatment should be provided if the load is not included in the plant design or if the side stream will upset the treatment process. Equalization of side streams should be considered to reduce instantaneous loading on the treatment process.

12.1.4 Multiple Units

Multiple units and/or storage facilities should be provided so that individual units may be taken out of service without unduly interrupting plant operation.

12.1.5 Sludge Pumps

12.1.5.1 Capacity

Pump capacities should be adequate to maintain pipeline velocities of 3 feet per second. Provisions for varying pump capacity are desirable.

12.1.5.2 Duplicate Units

Duplicate units shall be provided where failure of one unit would seriously hamper plant operation.

12.1.5.3 Type

Plunger pumps, progressing cavity pumps, or other types of pumps with demonstrated solids handling capability should be provided for handling raw sludge.

12.1.5.4 Minimum Head

A minimum positive head of 24 inches (or the manufacturer's recommendation) should be provided at the suction side of centrifugal-type pumps and is desirable for all types of sludge pumps. Maximum suction lifts should not exceed 10 feet (or the manufacturer's recommendation) for plunger pumps.

12.1.5.5 Sampling Facilities

Unless sludge sampling facilities are otherwise provided, quick-closing sampling valves should be installed at the sludge pumps. The size of valve and piping should be at least 1-1/2 inches.

12.1.6 Sludge Piping

12.1.6.1 Size and Head

Sludge withdrawal piping shall have a minimum diameter of 8 inches for gravity withdrawal and 6 inches for pump suction and discharge lines. Where withdrawal is by gravity, the available head on the discharge pipe should be at least 2 feet and preferably more, with provisions to backflush the line.

12.1.6.2 Slope

Gravity piping shall be laid on uniform grade and alignment. Slope on gravity discharge piping should not be less than 3 percent.

12.1.6.3 Cleaning

Provision should be made for draining and flushing suction and discharge lines. Where sludge pumps are available, piping should be such that suction lines can be backflushed with pump discharge or rodded. Glass-lined or equivalent pipe should be considered for raw sludge piping and scum lines.

12.1.6.4 Corrosion Resistance

Special consideration shall be given to the corrosion resistance and continuing stability of pipes and supports located inside digestion tanks.

12.2 <u>Sludge Production</u>

______The sludge production rates listed in the literature have often been shown to be underestimated. The sludge production rates (SPR) listed below in Table 12-1 have been determined from various studies and provide a more realistic basis for designing solids handling facilities. These values shall be used for design unless other acceptable data is submitted.

Table 12-1
Sludge Production Rates

Type of Treatment	SPR (lb sludge / lb BOD removed)
Conventional Activated Sludge	0.85
Extended Aeration	0.75
Contact Stabilization	1.00
Other Activated Sludge	0.85
Trickling Filter	0.75
Roughing Filters	1.00

12.3 Thickening

12.3.1 General

The cost-effectiveness of sludge thickening should be considered prior to treatment and/or disposal.

12.3.1.1 Capacity

Thickener design should provide adequate capacity to meet peak demands.

12.3.1.2 Septicity

Thickener design should provide means to prevent septicity during the thickening process. Odor consideration should be considered.

12.3.1.3 Continuous Return

Thickeners should be provided with a means of continuous return of supernatant for treatment. Provisions for side-stream treatment of supernatant may be required.

12.3.1.4 Chemical Addition

Consideration should be given to the use of chemicals or polymer to improve solids capture in the thickening process. This will not normally increase the solids level of the thickened sludge.

12.3.2 Gravity Thickeners

12.3.2.1 Stirring and skimming

Mechanical thickeners should employ pickets on rake arms for continuous gentle stirring of the sludge. Skimmers should be considered for use with biological sludges.

12.3.2.2 Depth and Freeboard

Tank depth shall be sufficient so that solids will be retained for a period of time needed to thicken the sludge to the required concentration and to provide storage for fluctuations in solids loading rates. The thickener should be operated to avoid denitrification.

At least two feet of freeboard shall be provided above the maximum water level.

12.3.2.3 Continuous Thickening

Variable-speed sludge draw-off pumps may be provided so that thickening can be continuous, or an adjustable on-off time clock control for pulse withdrawal may be used with constant-speed pumps to improve control over the thickening.

12.3.2.4 Solids and Surface Loading Rates

The engineer shall provide the design basis and calculations for the solids and surface loading rates and the support calculations upon request. Thickener solids loading rates vary with the type of sludge.
Some typical solids loading rates are given below in Table 12-2. These values shall be used for design unless other acceptable data are submitted. For loading rates of other type sludges, refer to Table 5.2 of the EPA Process Design Manual-Sludge Treatment and Disposal.

Table 12-2
Solids Loading Rate

Type of Sludge	Solids Loading Rate (lb/day/sq ft)
Primary	20-30
Activated sludge	5-6
Trickling filter	8-10
Primary and activated combined	6-10
Primary and trickling filter combined	10-12

Surface loading rates of 400 gallons per day per square foot (gpd/sq ft) or less will normally result in septic conditions. To prevent septic conditions, surface overflow rates should be maintained between 500 and 800 gpd/sq ft. For very thin mixtures or WAS only, hydraulic loading rates of 100-200 gpd/sq ft are appropriate. An oxygen-rich water source, such as secondary effluent, shall be available as a supplemental flow to the thickener to achieve the necessary overflow rates.

The diameter of a gravity thickener should not exceed 80 feet.

12.3.2.5 Bottom Slope

Bottom slopes shall be sufficient to keep the sludge moving toward the center well with the aid of a rake. Generally, the slope should be greater than conventional clarifiers. A floor slope of 2-3 inches per foot is recommended.

12.3.3 Flotation Thickeners

Flotation thickeners are normally used to concentrate waste activated sludge.

12.3.3.1 Air-Charged Water

The thickener underflow is generally used as a source of water for the air-charging units, although primary tank effluent or plant effluent may also be used.

12.3.3.2 Design Sizing

The engineer shall provide the design basis for sizing the units and for the support calculation. Design sizing should be based on rational calculations, including: total pounds of waste sludge anticipated, design solids and hydraulic loading of the unit, operating cycle in hours per day per week, removal efficiency, and quantity and type of chemical aids required. Flotation thickeners are normally sized by solids surface loadings. Typical design loadings range from 1.0 to 2.5 pounds per hour per square foot. (See Table 12-3, for typical solids loading rates to produce a minimum 4% solids concentration.)

12.3.3.3 Hydraulic Loading Rates

If polymers are used, hydraulic loading rates of 2.5 gpm/sq ft or less should be used. The hydraulic loading rates shall be lower if polymers are not used. Hydraulic loading rates shall be based on the total flow (influent plus recycle). The design of any thickened sludge pump from DAF units should be conservative. Frequently, polymer conditioned sludge will result in a solids concentration greater than 4%. Pumps shall be capable of handling a sludge of at least 5% thickness.

TABLE 12-3
TYPICAL DAF THICKENER SOLIDS LOADING RATES NECESSARY TO PRODUCE A MINIMUM 4 PERCENT SOLIDS CONCENTRATION

	Solids loading rate, lb/sq ft/hr	
Type of sludge	No chemical addition	Optimum chemical addition
Primary only	0.83 - 1.25	up to 2.5
Waste activated sludge (WAS)		
Air	0.42	up to 2.0
Oxygen	0.6 - 0.8	up to 2.2
Trickling filter	0.6 - 0.8	up to 2.0
Primary + WAS (air)	0.6 - 1.25	up to 2.0
Primary + trickling filter	0.83 - 1.25	up to 2.5

______12.3.4 Centrifugal Thickeners

12.3.4.1 Pretreatment

Any pretreatment required is in addition of that required for the main wastewater stream. For example, separate and independent grit removal may be needed for the centrifuge feed stream.

Disc nozzle centrifuges require pretreatment of the feed stream. Both screening and grit removal are required to reduce operation and maintenance requirements. Approximately 11% of the feed stream will be rejected in pretreatment, consideration should be given to the treatment of this flow. It is usually routed to the primary clarifier.

Basket centrifuges do not require pretreatment and are recommended in small plants (1.0-2.0 MGD) without primary clarification and grit removal.

Solid bowl decanter centrifuges require grit removal in the feed stream and are a potentially high maintenance item.

12.3.4.2 Chemical Coagulants

Provisions for the addition of coagulants to the sludge should be considered for improving dewatering and solids capture.

12.3.4.3 Design Data

The engineer shall provide the design basis for loading rates and support calculations. Both hydraulic and solids loading rate limitations should be addressed.

12.3.5 Other Thickeners

Other thickner designs will be evaluated on a case-by-case basis. Pilot plant data shall be provided by the design engineer upon request.

12.4 <u>Conditioning</u>

______12.4.1 General

Pretreatment of the sludge by chemical or thermal conditioning should be investigated to improve the thickening, dewatering, and/or stabilization characteristics of the sludge.

The effects of conditioning on downstream processes and subsequent side-stream treatment should be evaluated. Thermal conditioning will concentrate the BOD level of the side stream. Its treatment must be considered in calculating organic loadings of other units.

12.4.2 Chemical

Type of chemical, location of injection, and method of mixing should be carefully considered to ensure obtaining anticipated results. Pilot testing is often necessary to determine the best conditioning system for a given sludge.

12.5 Digestion

12.5.1 Anaerobic Digestion

12.5.1.1 General

a. Operability

Anaerobic digestion is a feasible stabilizing method for wastewater sludges that have low concentrations of toxins and a volatile solids content above 50%. It should not be used where wide variations in sludge quantity and quality are common. Anaerobic digestion is a complex process requiring close operator control. The process is very susceptible to upsets as the microorganisms involved are extremely sensitive to changes of their environment. Frequent monitoring of the following parameters is required:

(i) pH (6.4 - 7.5 recommended)

(ii) volatile acids/alkalinity ratio (always 0.5 or greater)

(iii) toxics (volatile acids, heavy metals, light metal cations, oxygen, sulfides, and ammonia)

(iv) temperature (within 1° F of design temperature)

(v) recycle streams (BOD, SS, NH_3, phenols)

The importance of avoiding digester upsets cannot be overlooked. The methane-producer bacteria have a very slow growth rate and it will take two weeks or more to resume normal digester performance.

b. Multiple Units

Multiple units should be provided. Staged digestion design may be used, provided the units can be used in parallel as well as in series. Where multiple units are not provided, a lagoon or storage tanks should be provided for emergency use so that digestion tanks may be taken out of service without unduly interrupting plant operation. Means of returning sludge from the secondary digester unit to the primary digester should be provided. In large treatment plants where digesters are provided, separate digestion of primary sludges
should be considered.

c. Depth

The proportion of depth to diameter should provide for the formation of a supernatant liquor with a minimum depth of 6 feet. Sidewall depth is generally about one-half the diameter of the digester for diameters up to 60 feet, and decreases to about one-third the diameter for diameters approaching 100 feet.

d. Maintenance Provisions

To facilitate emptying, cleaning, and maintenance, the following features are required:

(i) Slope

The tank bottom shall slope to drain toward the withdrawal pipe. A slope of between 1 inch per foot and 3 inches per foot is recommended.

(ii) Access Manholes

At least two access manholes should be provided in the top of the tank, in addition to the gas dome. One opening should be large enough to permit the insertion of mechanical equipment to remove scum, grit, and sand. A separate side wall manhole should be provided at ground level.

(iii) Safety

Nonsparking tools, rubber-soled shoes, safety harness, gas detectors for flammable and toxic gasses and the hose type or self-contained type breathing apparatus shall be provided.

e. Pre-thickening of sludge may be advantageous, but the solids content shall be less than 8% to ease mixing problems.

12.5.1.2 Sludge Inlets and Outlets

Multiple sludge inlets and draw-offs and multiple recirculation suction and discharge points should be provided to facilitate flexible operation and effective mixing of the digester contents, unless adequate mixing facilities are provided within the digester. One inlet should discharge above the liquid level and be located at approximately the center of the tank to assist in scum breakup. Raw sludge inlet points should be located to minimize short-circuiting to the supernatant drawoff.

12.5.1.3 Tank Capacity

a. General

Two cultures of bacteria are primarily involved in anaerobic digestion: acid formers and methane formers. Capacity of the digester tank shall be based on the growth rate of the methane-formers, as they have extremely slow growth rates.

b. Solids Basis

Where the composition of the sewage has been established, tank capacity should be computed from the volume and character of sludge to be digested. The total digestion tank capacity should be determined by rational calculations based upon factors such as volume of sludge added, its percent solids and character, volatile solids loading, temperature to be maintained in the digesters, and the degree or extent of mixing to be obtained. These detailed calculations shall be submitted to justify the basis of design.

Where composition of the sewage has not been established, the minimum combined digestion tank capacity outlined below shall be provided. Such requirements assume that the raw sludge is derived from ordinary domestic wastewater, a digestion temperature is maintained in the range of 85° to 100° F, there is 40 to 50 percent volatile matter in the digested sludge, and that the digested sludge will be removed frequently from the process.

(i) Completely Mixed Systems

For heated digestion systems providing for intimate and effective mixing of the digester designed for a constant feed loading rate of 150 to 400 pounds 1,000 cubic feet of volume per day in the active digesting unit. The design average detention time in completely mixed systems shall have sufficient mixing capacity to provide for complete digester turnover every 30 minutes.

(ii) Moderately Mixed Systems

For digestion systems where mixing is accomplished only by circulating external heat exchanger, the system may be loaded up to 40 pounds of volatile solids per 1,000 cubic feet of volume per day in the active digestion units. This loading may be modified upward or downward, depending upon the degree of mixing provided. Where mixing is accomplished by other methods, loading rates will be determined on the basis of information furnished by the design engineer.

c. Population Basis

Where solids data are not available, the following unit capacities shown in Table 12-4 for conventional, heated tanks shall be used for plants treating domestic sewage. The capacities should be increased by allowing for the suspended solids population equivalent of any industrial wastes in the sewage. The capacities stated apply where digested sludge is dewatered on sand drying beds and may be reduced if the sludge is dewatered mechanically or otherwise frequently withdrawn.

Table 12-4
Cubic Feet Per Capita

Type of Plant	Moderately Mixed Systems	Completely Mixed Systems
Primary	2 to 3	1.3
Primary and Trickling Filter	4 to 5	2.7 to 3.3

Primary and Activated Sludge	4 to 6	2.7 to 4

For small installations (population 5,000 or less) the larger values should be used.

12.5.1.4 Gas Collection System

a. General

All portions of the gas system, including the space above the tank liquor, storage facilities, and piping shall be so designed that under all normal operating conditions, including sludge withdrawal, the gas will be maintained under positive pressure. All enclosed areas where any gas leakage might occur shall be adequately ventilated.

b. Safety Equipment

All necessary safety facilities shall be included where gas is produced. Pressure and vacuum relief valves and flame traps, together with automatic safety shutoff valves, are essential. Water-seal equipment shall not be installed on gas piping.

c. Gas Piping and Condensate

Gas piping shall be of adequate diameter and shall slope to condensation traps at low points. The use of float-controlled condensate traps is not permitted. Condensation traps shall be placed in accessible locations for daily servicing and draining. Cast iron, ductile iron, and/or stainless steel piping should be used.

d. Electrical Fixtures and Equipment

Electrical fixtures and equipment in enclosed places where gas may accumulate shall comply with the National Board of Fire Underwriters' specifications for hazardous conditions. Explosion-proof electrical equipment shall be provided in sludge-digestion tank galleries containing digested sludge piping or gas piping and shall be provided in any other hazardous location where gas or digested sludge leakage is possible.

e. Waste Gas

Waste gas burners shall be readily accessible and should be located at least 50 feet away from any plant structure, if placed near ground level, or may be located on the roof of the control building if sufficiently removed from the tank. Waste gas burners shall not be located on top of the digester. The waste gas burner should be sized and designed to ensure complete combustion to eliminate odors.

f. Ventilation and Cover

Any underground enclosures connecting with digestion tanks or containing sludge or gas piping or equipment shall be provided with forced ventilation. Tightly fitting, self-closing doors shall be provided at connecting passageways and tunnels to minimize the spread of gas. A
floating cover should be provided instead of a fixed cover for increased operational flexibility and safety.

g. Metering

Gas meters with bypasses should be provided to meter total gas production and utilization.

h. Pressure Indication

Gas piping lines for anaerobic digesters should be equipped with closed-type pressure indicating gauges. These gauges should read directly in inches of water. Normally, three gauges should be provided, one to measure the main line pressure, a second to measure the pressure upstream of gas-utilization equipment, and the third to measure pressure to wasteburners. Gas-tight shutoff and vent cocks shall be provided. The vent piping shall be extended outside the building, and the opening shall be screened to prevent entrance by insects and turned downward to prevent entrance of rainwater. All piping shall be protected with safety equipment.

i. Gas Utilization Equipment

Gas-burning boilers, engines, and other gas utilization equipment should be located at or above ground level in well-ventilated rooms. Gas lines to these units shall be provided with suitable flame traps.

12.5.1.5 Heating

a. Insulation

Digestion tanks should be constructed above the water table and should be suitably insulated to minimize heat loss.

b. Heating Facilities

Sludge may be heated by circulating the sludge through external heaters or by units located inside the digestion tank.

(i) External Heating

Piping should be designed to provide for the preheating of feed sludge before introduction to the digesters. Provisions
should be made in the layout of the piping and valving to facilitate cleaning of these lines.

Heat exchanger sludge piping should be sized for heat transfer requirements.

(ii) Internal Coils

Hot water coils for heating digestion tanks should be at least 2 inches in diameter and the coils, support brackets, and all fastenings should be of corrosion-resistant material. The use of dissimilar metals should be avoided to minimize galvanic action. The high point in the coils should be vented to avoid air lock.

(iii) Other Methods

Other types of heating facilities will be considered on their own merits.

c. Heating Capacity

Sufficient heating capacity shall be provided to consistently maintain the digesting sludge temperature to within 1°F (0.6°C) of the design temperature. An alternate source of fuel should be available and the boiler or other heat source should be capable of using the alternate fuel if digester gas is the primary fuel. Thermal shocks shall be avoided. Sludge storage may be required to accomplish this.

d. Hot Water Internal Heating Controls

(i) Mixing Valves

A suitable automatic mixing valve should be provided to temper the boiler water with return water so that the inlet water to the heat jacket or coils can be held to below a temperature (130° to 150°F) at which sludge caking will be accentuated. Manual control should also be provided by suitable bypass valves.

(ii) Boiler Controls

The boiler should be provided with suitable automatic controls to maintain the boiler temperature at approximately 180°F to minimize corrosion and to shut off the main fuel supply in the event of pilot burner or electrical failure, low boiler water level, or excessive temperature.

(iii) Thermometers

Thermometers shall be provided to show temperatures of the sludge, hot water feed, hot water return, and boiler water.

12.5.1.6 Mixing

Facilities for mixing the digester contents shall be provided where required for proper digestion by reason of loading rates, or other features of the system.

12.5.1.7 Supernatant Withdrawal

a. Piping Size

Supernatant piping should not be less than 6 inches in diameter, although 4-inch lines will be considered in special cases.

b. Withdrawal Arrangements

(i) Withdrawal Levels

Piping should be arranged so that withdrawal can be made from three or more levels in the tank. A positive unvalved vented overflow shall be provided.

(ii) Withdrawal Selection

On fixed-cover tanks the supernatant withdrawal level should preferably be selected by means of interchangeable extensions at the discharge end of the piping.

(iii) Supernatant Selector

If a moveable supernatant selector is provided, provision should be made for at least one other draw-off level located in the supernatant zone of the tank in addition to the unvalved emergency supernatant draw-off pipe. High-pressure backwash facilities should be provided.

c. Sampling

Provisions shall be made for sampling at each supernatant draw-off level. Sampling pipes should be at least 1-1/2 inches in diameter.

d. Supernatant Handling

Problems such as shock organic loads, pH, and high ammonia levels associated with digester supernatant shall be addressed in the plant design. Recycle streams should be bled continuously back to the treatment process.

12.5.2 Aerobic Sludge Digestion

12.5.2.1 Mixing and Aeration

Aerobic sludge digestion tanks shall be designed for effective mixing and aeration. Minimum mixing requirements of 20 cubic feet per minute per 1,000
cubic feet for air systems and 0.5 horsepower per 1,000 cubic feet for mechanical systems are recommended. Aeration requirements may be more or less than the mixing requirements, depending on system design and actual solids loading. Approximately 2.0 pounds of oxygen per pound volatile solids are needed for aeration. If diffusers are used, types should be provided to minimize clogging and designed to permit removal for inspection,

maintenance, and replacement without dewatering the tanks, if only one digester is proposed.

12.5.2.2 Size and Number of Tanks

The size and number of aerobic sludge digestion tank or tanks should be determined by rational calculations based upon such factors as volume of sludge added, its percent solids and character, the degree of volatile solids reduction required and the size of installation with appropriate allowance for sludge and supernatant storage.

Generally, 40 to 50 percent volatile solids destruction is obtained during aerobic digestion. To ensure a stabilized sludge which will not emit odors, the volatile solids content should be less than 60 percent in the digested sludge. Calculations shall be submitted upon request to justify the basis of design. The following design parameter ranges should be considered the minimum in designing aerobic digestion facilities.

a. Hydraulic Detention Time

Hydraulic detention time at 20°C should be in the range of 15 to 25 days, depending upon the type of sludge being digested. Activated sludge alone requires the lower detention time and a combination of primary plus activated or trickling filter sludges requires the high detention time. Detention times should be adjusted for operating temperatures other than 20°C.

b. Volatile Solids

The volatile solids loading shall be in the range of 0.1 to 0.2 pound of volatile solids per cubic foot per day.

c. Dissolved Oxygen

Design dissolved oxygen concentration should be in the range of 1 to 2 mg/l. A minumum of 1.0 mg/l shall be maintained at all times.

d. Mixing Energy

Energy input requirements for mixing should be in the range of 0.5 to 1.5 horsepower per 1,000 cubic feet where mechanical aerators are used; 20 to 35 standard cubic feet of air per minute per 1,000 cubic feet of aeration tank where diffused air mixing is used on activated sludge alone; and greater than 60 cubic feet per minute per 1,000 cubic feet for primary sludge alone and primary plus activated sludge.

e. Storage

Detention time should be increased for temperatures below 20°C. If sludge cannot be withdrawn during certain periods, additional storage capacity should be provided. Plants smaller than 75,000 gpd should have storage capacity of 2 cubic foot per population equivalent served.

12.5.2.5 Supernatant Separation

Facilities should be provided for separation or decantation of supernatant. Provisions for sidestream treatment of supernatant should be considered.

12.6 Composting

______Composting operations will be considered on a case-by-case basis, provided that the basis for design and a cost-effective analysis are submitted by the engineer.

12.7 Sludge Dewatering

______12.7.1 General

Drainage from drying beds and centrate or filtrate from dewatering units should be returned to the sewage treatment process at appropriate points preceding the secondary process. The return flows shall be returned downstream of the influent sample and/or flow measuring point and a means shall be provided to sample return flows. These organic loads must be considered in plant design.

12.7.2 Sludge Drying Beds

12.7.2.1 Area

It is recommended that wastewater systems have a hybrid sludge disposal method because of the seasonal downtime associated with drying beds. The amount of rainfall
normal for our state makes the use of sludge drying beds insufficient at times.

Consideration shall be given to the location of drying beds to avoid areas where moisture in the air is higher than normal (i.e., adjacent to rivers where morning fog is common).

In determining the area for sludge drying beds, consideration shall be given to climatic conditions, the character and volume of the sludge to be dewatered, type of bed used, and methods of ultimate sludge disposal. Design calculations shall be submitted upon request to substantiate the area used.

Drying bed design should be based on square feet per capita or pounds of sludge solids per square foot per year. Table 12-5 presents the range of values that should be used, these values are for drying anaerobically digested sludges. Additional area is required for wetter sludges such as those resulting from aerobic digestion; therefore, use the higher number of the required range.

Table 12-5 DRYING BED DESIGN CRITERIA*

	Open Beds		Covered Beds
Type of Sludge	Per Capita (sq ft/capita)	Solids (lb/sq ft/yr)	Per Capita (sq ft/capita)
Primary	1.0 to 1.5	27.5	0.75 to 1.0

Attached Growth	1.25 to 1.75	22.0	1.0 to 1.25
Suspended Growth	2.50	15.0	2.00

*The design engineer should rely on his experience and the plant location.

These criteria are a minimum.

____________12.7.2.2. Percolation Type

a. Gravel

The lower course of gravel around the underdrains should be properly graded to range in size from 1/4-inch to 1-inch and should be 12 inches in depth, extending at least 6 inches above the top of the underdrains. It is desirable to place this in 2 or more layers. The top layer of at least 3 inches should consist of gravel 1/8 inch to 1/4 inch in size. The gravel shall be laid on an inpervious surface so that the filtrate will not escape to the soil.

b. Sand

The top course shall consist of at least nine inches of sand with a uniformity coefficient of less than 3.5. For trickling filter sludge, the effective size of the sand shall be between 0.8 to 3.0 millimeter. For waste activated sludge, the effective size of the sand shall be between 0.5 to 0.8 millimeter. For combinations, use the lower size range.

c. Underdrains

Underdrains should be clay pipe, concrete drain tile, or other underdrain acceptable material and shall be at least 4 inches in diameter and sloped not less than 1 percent to drain. Underdrains shall be spaced between 8 and 20 feet apart. The bottom of the bed shall slope towards the underdrains. Consideration should be given to placing the underdrain in a trench.

12.7.2.3 Impervious Types

Paved surface beds may be used if supporting data to justify such usage are acceptable to the Department. The use of paved beds for aerobically digested sludge is generally not recommended.

12.7.2.4 Walls

Walls should be watertight and extend 15 to 18 inches above the ground surface. Outer walls should be curbed to prevent soil from washing onto the beds.

12.7.2.5 Sludge Removal

Not less than two beds should be provided and they should be arranged to facilitate sludge removal. Concrete truck tracks should be provided for all percolation-type sludge beds with pairs of tracks for the beds on appropriate centers. If truck access is by way of an opening in the drying bed wall, the opening shall be designed so that no sludge will leak out during the filling process.

12.7.2.6 Sludge Influent

The sludge pipe to the beds should terminate at least 12 inches above the surface and be arranged so that it will drain. Concrete splash plates shall be provided at sludge discharge points.

12.7.3 Mechanical Dewatering

12.7.3.1 Methods and Applicability

The methods used to dewater sludge may include use of one or more of the following devices:

a. Rotary vacuum filters

b. Centrifuges, either solid bowl or basket type

c. Filter presses

d. Horizontal belt filters

e. Rotating gravity concentrators

f. Vacuum drying beds

g. Other "media type" drying beds

The technology and design of sludge dewatering devices are constantly under development; therefore, each type should be given careful consideration. The applicability of a given method should be determined on a case-by-case basis, with the specifics of any given situation being carefully evaluated, preferably in pilot tests. The engineer shall justify the method selected using pilot plant data or experience at a similar treatment plant.

12.7.3.2 Considerations

Considerations in selection should include:

a. Type and amount of sludge

b. Variations in flow rate and solids concentration

c. Capacity of the equipment

d. Chemicals required for conditioning

e. Degree of dewatering required for disposal

f. Experience and qualifications of plant staff

g. Reliability

h. Operation and maintenance cost

i. Space requirements

12.7.3.3 Storage

Adequate storage shall be provided for all systems.

12.8 Sludge Storage Lagoons

______Refer to Chapter 9, Ponds and Aerated Lagoons, for the requirements of sludge storage lagoons.

12.9 Sludge Disposal

______The ultimate disposal of sludge through various methods (i.e., landfilling, land application) is subject to the regulations and/or guidelines of the Tennessee Division of Water Pollution Control (DWPC). Approval by DWPC is required prior to initiation of the selected disposal alternative.

CHAPTER 13

Plant Flow Measurement and Sampling

13.1 Purpose

13.2 Flow Measurement

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- 13.2.2 Parshall Flumes
- 13.2.3 Sharp Crested Weirs
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13.3 Sampling

- 13.3.1 Automatic Sampling Equipment
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PLANT FLOW MEASUREMENT AND SAMPLING

13.1 Purpose

Complete and accurate flow measuring and sampling are essential in the proper treatment of wastewater. Compliance with discharge limits requires proper flow measurement and sampling. They provide the operator with the information to optimize process control and operational costs, as well as providing an accurate data base of flows and process performance which can be used to analyze changes in operational strategy or assist future plant design.

13.2 Flow Measurement

13.2.1 General Considerations

13.2.1.1 Facilities for measuring the volume of sewage flows should be provided at all treatment works.

13.2.1.2 Plants with a capacity equal to or less than 100,000 gallons per day (gpd) shall be equipped, as a minimum, with a primary metering device such as: a Parshall flume having a separate float well and staff gauge, a weir box having plate and staff gauge, or other approved devices. Continuous recording devices may be required where circumstances warrant.

13.2.1.3 Plants having a capacity of greater than 100,000 gpd shall be provided with indicating, recording, and totalizing equipment using strip or circular charts and with flow charts for periods of 1 or 7 days. The chart size shall be sufficient to accurately record and depict the flow measured.

13.2.1.4 Flows passed through the plant and flows bypassed shall be measured in a manner which will allow them to be distinguished and separately reported.

13.2.1.5 Measuring equipment shall be provided which is accurate under all expected flow conditions (minimum initial flow and maximum design peak flow). The accuracy of the total flow monitoring system (primary device, transmitter, and indicator) must be acceptable. The effect of such factors as ambient temperature, power source voltage, electronic interference, and humidity should be considered. Surges must be eliminated to provide accurate measurement. Two primary devices and flow charts may be required in some cases.

13.2.1.6 Metering devices within a sewage works shall be located so that recycle flow streams do not inadvertently affect the flow measurement. In some cases,
measurement of the total flow (influent plus recycle) may be desirable.

13.2.1.7 All clarifiers must be provided with a means for accurate flow measurement of sludge wasting and sludge return lines so that solids handling can be controlled. Sludge digesters, thickeners, and holding tanks should be provided with some way to determine the volume of sludge added or removed. This can be accomplished by a sidewall depth scale or graduation in batch operations.

13.2.1.8 Flow meter and indicator selection should be justified considering factors such as probable flow range, acceptable headloss, required accuracy, and fouling ability of the water to be measured. For more detailed information the consultant is encouraged to read the EPA Design Information Report "Flow Measurement Instrumentation"; Journal WPCF, Volume 58, Number 10, pp. 1005-1009. This report offers many installation details and considerations for different types of flow monitoring equipment.

13.2.1.9 Flow splitter boxes shall be constructed so that they are reliable, easily controllable, and accessible for maintenance purposes.

13.2.1.10 Where influent and effluent flow-proportional composite sampling is required, separate influent and effluent flow measuring equipment is required.

13.2.1.11 Consideration should be given to providing some types of flow meters with bypass piping and valving for cleaning and maintenance purposes.

13.2.2 Parshall Flumes

Parshall Flumes are ideal for measuring flows of raw sewage and primary effluents because clogging problems are usually minimal.

The properly sized flume should be selected for the flow range to be encountered. All Parshall Flumes must be designed to the specified dimensions of an acceptable reference.

The following requirements must be met when designing a Parshall Flume.

13.2.2.1 Flow should be evenly distributed across the width of the channel.

13.2.2.2 The crest must have a smooth, definite edge. If a liner is used, all screws and bolts should be countersunk.

13.2.2.3 Longitudinal and lateral axes of the crest floor must be level.

13.2.2.4 The location of the head measuring points (stilling well) must be two-thirds the length of the converging sidewall upstream from the crest. Sonar-type devices are only acceptable when foaming or turbulance is not a problem.

13.2.2.5 The pressure tap to the stilling well must be at right angles to the wall of the converging section.

13.2.2.6 The invert (i.e., inside bottom) of the pressure tap must be at the same elevation as the crest.

13.2.2.7 The tap should be flush with the flume side wall and have square, sharp corners free from burrs or other projections.

13.2.2.8 The tap pipe should be 2 inches in size and be horizontal or slope downward to the stilling well.

13.2.2.9 Free-flow conditions shall be maintained under all flow rates to be encountered by providing low enough elevations downstream of the flume. No constrictions (i.e., sharp bends or decrease in pipe size) should be placed after the flume as this might cause submergence under high flow conditions.

13.2.2.10 The volume of the stilling well should be determined by the conditions of flow. For flows that vary rapidly, the volume should be small so that the instrument float can respond quickly to the changes in rate. For relatively steady flows, a large-volume stilling well is acceptable. Consideration should be given to protecting the stilling well from freezing.

13.2.2.11 Drain and shut-off valves shall be provided to empty and clean the stilling well.

13.2.2.12 Means shall be provided for accurately maintaining a level in the stilling well at the same elevation as the crest in the flume, to permit adjusting the instrument to zero flow conditions.

13.2.2.13 The flume must be located where a uniform channel width is maintained ahead of the flume for a distance equal to or greater than fifteen (15) channel widths. The approach channel must be straight and the approaching flow must not be turbulent, surging, or unbalanced. Flow -lines should be essentially parallel to the centerline of the flume.

13.2.3 Sharp Crested Weirs

The following criteria are for V-notch weirs, rectangular weirs with and without end contractions, and Cipolletti weirs. The following details must be met when designing a sharp crested weir:

13.2.3.1 The weir must be installed so that it is perpendicular to the axis of flow. The upstream face of the bulkhead must be smooth.

13.2.3.2 The thickness of the weir crest should be less than 0.1 inch or the downstream edge of the crest must be relieved by chamfering at a 45° angle so that the horizontal (unchamfered) thickness of the weir is less than 0.1 inch.

13.2.3.3 The sides of rectangular contracted weirs must be truly vertical. Angles of V-notch weirs must be cut precisely. All corners must be machined or filed perpendicular to the upstream face so that the weir will be free of burrs or scratches.

13.2.3.4 The distance from the weir crest to the bottom of the approach channel must be greater than twice the maximum weir head and is never to be less than one foot.

13.2.3.5 The distance from the sides of the weir to the side of the approach channel must be greater than twice the maximum weir head and is never to be less than one foot (except for rectangular weirs without end contractions.)

13.2.3.6 The nappe (overflow sheet) must touch only the upstream edges of the weir crest or notch. If properly designed, air should circulate freely under and on both sides of the nappe. For suppressed rectangular weirs (i.e., no contractions), the enclosed space under the nappe must be adequately ventilated to maintain accurate head and discharge relationships.

13.2.3.7 The measurement of head on the weir must be taken at a point at least four (4) times the maximum head on the crest upstream from the weir.

13.2.3.8 The cross - sectional area of the approach channel must be at least eight (8) times that of the nappe at the crest for a distance upstream of

15-20 times the maximum head on the crest in order to minimize the approach velocity. The approach channel must be straight and uniform upstream of the weir for the same distance, with the exception of weirs with end contractions where a uniform cross section is not needed.

13.2.3.9 The head on the weir must have at least three (3) inches of free fall at the maximum downstream water surface to ensure free fall and aeration of the nappe.

13.2.3.10 All of the flow must pass over the weir and no leakage at the weir plate edges or bottom is permissable.

13.2.3.11 The weir plate is to be constructed of a material equal to or more resistant than 304 Stainless Steel.

13.2.4 Venturi and Modified Flow Tube Meters

The following requirements should be observed for application of venturi meters:

13.2.4.1 The range of flows, hydraulic gradient, and space available for installation must be suitable for a venturi meter and are very important in selecting the mode of transmission to the indicator, recorder, or totalizer.

13.2.4.2 Venturi meters shall not be used where the range of flows is too great or where the liquid may not be under a positive head at all times.

13.2.4.3 Cleanouts or handholes are desirable, particularly on units handling raw sewage or sludge.

13.2.4.4 Units used to measure air delivered by positive - displacement blowers should be located as far as possible from the blowers, or means should be provided to dampen blower pulsations.

13.2.4.5 The velocity and direction of the flow in the pipe ahead of the meter can have a detrimental effect on accuracy. There should be no bends or other fittings for 6 pipe diameters upstream of the venturi meter, unless treated effluent is being measured when straightening vanes are provided.

13.2.4.6 Other design guidelines as provided by manufacturers of venturi meters should also be considered.

13.2.5 Other Flow Metering Devices

Flow meters, such as propeller meters, magnetic flow meters, orifice meters, pitot tubes, and other devices, should only be used in applications in accordance with the manufacturer's recommendations and design guidelines.

13.2.6 Hydrograph Controlled Release (HCR) Systems

For plants utilizing HCR systems, accurate stream flow measurements are required. Detailed plans must be submitted outlining the construction of the primary stream flow measuring device and the associated instrumentation. The following factors should be emphasized in the design.

13.2.6.1 Accuracy over the flow range required for effluent discharge limiting purposes.

13.2.6.2 Operational factors such as cleaning and maintenance requirements.

13.2.6.3 Cost

The use of sharp crested weirs as described in Section 13.2.3 will not be allowed due to the installation requirements such as approach channel details and upstream pool depth and since entrapment and accumulation of silt and debris may cause the device to measure inaccurately. Parshall Flumes may be used due to their self-cleaning ability but field calibration will be required. Self-cleaning V-notch weirs are recommended due to their accuracy in low flow ranges. The weir can be made self-cleaning by sloping both sides of the weir away from the crest. The top portion of the crest shall be covered with angle-iron to prevent its breakdown. The angle of the V-notch should be determined by the stream characteristics; however, a smaller angle will increase accuracy in the low flow range. The primary device shall be built with sufficient depth into the stream bed to prevent undercutting and sufficient height to cover the required flow range.

It is recommended that the wastewater system director, engineer, or other city official contact the U.S. Geological Survey (USGS), Water Resources Division, in Nashville, Tennessee, for assistance with the design and installation of the flow measuring device. They offer a program which shares much of the costs for designing and maintaining the device. After visiting the site, they can assist with the design of a self-cleaning weir for the stream. They provide the consultant with a field design that shows the proper location and installation of the weir. From this field design, the consultant must provide detailed plans to the State. The wastewater system is responsible for constructing the weir at their own cost. The flow measuring station is installed, maintained, and calibrated by USGS personnel so that accurate results are insured. The primary device will record continuous flow of the stream and can be designed to send a feedback signal to the WWTP for other purposes such as controlling plant discharge rates. This program benefits both the local wastewater system, the State of Tennessee, and the USGS, as it adds to stream flow data bases archived for public use. Cost sharing allows the flow measuring station to be built and operated at a lower cost for all parties concerned.

13.3 Sampling

13.3.1 Automatic Sampling Equipment

The following general guidelines should be adhered to in the use of automatic samplers:

13.3.1.1 Automatic samplers shall be used where composite sampling is necessary.

13.3.1.2 The sampling device shall be located near the source being sampled, to prevent sample degradation in the line.

13.3.1.3 Long sampling transmission lines should be avoided.

13.3.1.4 If sampling transmission lines are used, they shall be large enough to prevent plugging, yet have velocities sufficient to prevent sedimentation. Provisions shall be included to make sample lines cleanable. Minimum velocities in sample lines shall be 3 feet per second under all operating conditions.

13.3.1.5 Samples shall be refrigerated unless the samples will not be effected by biological degradation.

13.3.1.6 Sampler inlet lines shall be located where the flow stream is well mixed and representative of the total flow.

13.3.1.7 Influent automatic samplers should draw a sample downstream of bar screens or comminutors. They should be located before any return sludge lines or scum lines.

13.3.1.8 Effluent sampling should draw a sample immediately upstream of the chlorination point. This will eliminate the need to dechlorinate and then re-seed the sample.

13.3.2 Manual Sampling

Because grab samples are manually obtained, safe access to sampling sites should be considered in the design of treatment facilities.

13.3.3 Long Outfall Lines

Many wastewater systems are constructing long outfall lines to take advantage of secondary or equivalent permit limits. Due to possible changes in effluent quality between the treatment facility and the outfall, a remote sampling station will be required at or near the confluence of the outfall line and the receiving stream on all outfall lines greater than one mile in length. Dissolved oxygen, fecal coliform, and chlorine residual may have to be measured at the remote sampling station for permit compliance purposes.

13.3.4 Sampling Schedules

Samples must be taken and analyzed for two purposes: permit compliance and process control. Any time a new permit is issued, a sampling schedule for permit compliance will be determined by the Division of Water Pollution Control. An additional sampling program needs to be set up for process control purposes. This would include all testing required for completing the monthly operational report, as well as any other tests that might aid the operation of the plant. This schedule can be determined by the Division of Water Pollution Control, Wastewater Treatment Section or the appropriate field office once final plans are approved. The designer shall provide safe access points to collect representative influent and effluent samples of all treatment units and to collect samples of all sludge transmission lines. This makes it possible to determine the efficiency of each treatment process. Additional information about methods of analyses can be obtained from the Federal Register 40 CFR Part 136. Information about sampling locations and techniques can be obtained from the EPA Aerobic Biological Wastewater Treatment Facilities Process Control Manual and EPA's NPDES Compliance Inspection Manual.

CHAPTER 14

Instrumentation, Control and Electrical Systems

INSTRUMENTATION, CONTROL AND ELECTRICAL SYSTEMS

14.1 General Requirements

14.1.1 Codes and Regulations

Sewage treatment systems are classified by reliability as noted in publication number EPA-430-99-74-001. Plant instrumentation, control and electrical systems shall be designed to comply with the applicable requirements of this standard. See Chapter 1, Section 1.3.11.

The design of the treatment facilities instrumentation, control and electrical systems shall conform to applicable codes and regulations including:

National Electric Code (NEC)
Occupational Safety and Health Act (OSHA)
State and Local Building Codes
National Electrical Safety Code (NESC)
Instrument Society of America (ISA)

14.1.2 Plan Requirements

The instrument and control plans shall include, as a minimum, the following drawings:

Instrumentation, control and systems legend and general notes

Process and instrumentation diagram (P&ID)

Process flow diagram (may be combined in P&ID)

Site plan

Plant power distribution plan (can be included in site plan)

Switching logic or schematic drawings

Complete electrical one-line diagram

Building lighting plans

Building power plans

Motor control diagram

Equipment and installations details as required

Instrument loop diagram

14.2 Instrumentation and Control Systems Requirements

14.2.1 General

An instrumentation and control system must be designed with both operational reliability (accurate and repeatable results) and maintainability if it is to properly serve its purpose.

14.2.2 Backup Equipment

Instrumentation whose failure could result in wastewater bypassing or a violation of the effluent limitations shall be provided with an installed backup sensor and readout. The backup equipment may be of a different type and located at a different point, provided that the same function is performed. No single failure shall result in disabling both sets of parallel instrumentation.

14.2.3 Automatic Control

Where system automation is employed, a manual intervention/override or backup shall be provided.

14.2.4 Calibration

Vital instrumentation and control equipment shall be designed to permit alignment and calibration without requiring bypassing of wastewater or a violation of the effluent limitations. Automated systems shall have provisions for operator verification of performance and all necessary systems calibration devices.

14.2.5 Test Circuits

Test circuits shall be provided to enable the alarms and annunciators to be tested and verified to be in working order.

14.2.6 Alarms and Annunciators

Alarms and annunciators shall be provided to monitor the condition of equipment whose failure could result in wastewater bypassing or a violation of the effluent limitations. Alarms and annunciators shall also be provided to monitor conditions which could result in damage to vital equipment or hazards to personnel. The alarms shall sound in areas normally manned and also in areas near the equipment. The combination of alarms and annunciators shall be such that each announced condition is uniquely identified.

14.3 Electrical System Requirements

14.3.1 Electric Power Sources

14.3.1.1 Primary Power Source

Generally, the local electric utility will be the primary source of electrical power. Second source of electrical power may be on-site generation or a second connection to the electric utility. If the second source is a connection to the electric utility, it must be arranged that a failure of one source does not directly effect the other. See Chapter 1, Reliability Class.

14.3.1.2 Standby Power Source

All treatment facilities greater than 100,000 gpd (average design flow) shall be equipped with an emergency generator to provide an alternate power source when a second power source is not available. The capacity of the backup emergency generator system shall conform to the Reliability Classification together with critical lighting and ventilation. If a main pump station is on site (or near) and would result in zero flow reaching the plant during power outages, it shall have a second power feed or standby power.

14.3.2 Power Distribution Within the Plant

The electrical power distribution system within the plant should be planned and designed on the following basis:

Plant electrical loads (peak and average demand)

Maximum fault currents available

Proper protective device coordination and device fault current withstand and interrupt ratings

Plant physical size and distribution of electrical loads

Plant power factor correction requirements

Location of other plant utility systems and facilities

Reliability requirements

Voltage drop limitations

Planned future plant expansions

Feasibility and possible economic justification for electrical demand control system

Life-cycle cost of major electrical equipment

All codes and regulations, and good engineering practice

14.4 Miscellaneous Requirements

14.4.1 Fire and Flooding

Failure of electrical equipment from such causes as fire and flooding shall be minimized by provision of suitable equipment housing and location, as well as by proper equipment design.

14.4.2 Housing of Electrical Equipment

Where practicable, electrical equipment shall be located in a separate room having an adequately controlled environment.

14.4.3 Ventilation

Mechanical ventilation shall be provided as necessary to protect electrical equipment from excessive temperatures.

14.4.4 Spare Components

An adequate number of spare components shall be specified by the design engineer to permit in-plant repairs or modifications and adjustment. These components include starters, low voltage contactors, and buried conduit. Spare electrical components which are subject to wear, such as motor brushes and switches, should also be specified by the design engineer as appropriate to minimize downtime.

14.4.5 Lighting

Adequate lighting throughout the wastewater treatment facility shall be provided, particularly in areas of operation and maintenance activities. Adequate emergency lighting shall be provided in the event of power failure.

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THIS IS CHAPTER 15

CHAPTER 15

Small Alternative Systems

SMALL ALTERNATIVE SYSTEMS

15.1 Preface

This chapter is prepared as a supplement to the main design criteria. This supplement attempts to elaborate on some of the critical considerations of small flow design which may modify or complement the general design criteria. This chapter presents the method to determine the proper design for small flows including the examination of new treatment alternatives. Small flows are defined as domestic wastewater flows from approximately 1,000 to 150,000 gallons per day.

15.2 General Considerations

Small treatment plants require different design considerations than larger plants. During the design of a small treatment facility, the design engineer shall evaluate the feasibility of various process alternatives (including subsurface disposal) to meet the design disposal requirements. An engineering report must be submitted to the Department of Environment and Conservation, Division of Water Pollution Control, detailing the method of determining the chosen treatment alternative. The engineering report shall present an economic analysis of alternative treatment types; both capital, operation and maintenance costs. The reliability of the treatment alternatives must be examined with respect to the sensitivity of the receiving stream for direct discharges and ground water protection for subsurface disposal. If subsurface disposal is eliminated due to soil or groundwater conditions, a letter from the Division of Ground Water Protection must accompany the engineering report. Thus, engineering and environmental judgments shall be used to balance the economy of construction and operations with the reliability of appropriate treatment alternatives based on the sensitivity of the site.

15.3 Selection Guidelines

The following steps shall be utilized to select the treatment scheme or alternative for each site. In general these are:

15.3.1 First, examine the possibility of transporting flows to a nearby wastewater treatment plant.

15.3.2 Exhaust the possibility of disposing of flows by subsurface disposal. For example, given favorable site conditions, up to 10,000 gpd may be disposed of in a low pressure pipe disposal field. Remote sites should be considered.

15.3.3 Exhaust the possibility of water conservation and/or reuse systems to limit the disposal flows. Examples include spray irrigation for laundry wastes and surface drip (Israel emitters) irrigation.

15.3.4 Exhaust the development of "passive" treatment systems such as lagoons, artificial wetlands or hydragraphic controlled release (HCR), which are less prone to mechanical and operational problems. HCR systems may be designed to discharge only during winter months or high flow conditions. HCR's must include automatic controls and stream flow measurement devices.

15.3.5 Exhaust the possibility of easy-to-operate mechanical systems such as recirculating sand filters (see Section 15.7.1 for specific criteria), as opposed to suspended media systems.

15.3.6 As a last resort, a mechanical package activated sludge plant may be used. Package activated sludge plants, however, will not be approved for flows below 50,000 gallons per day due to economic, operational and maintenance requirements.

15.3.7 DESIGN BY ANALOGY

Data from similar existing systems may be utilized in the case of new design concepts which have reliability and operability merit; however, thorough investigation and adequate documentation shall be made to establish the reliability and applicability of such data.

15.4 Planning

The applicant shall contact the appropriate Water Pollution Control Field Office as early as possible in the planning process. The proposed project shall be discussed and the applicant or design engineer will be advised of information required in submittals. The treatment works will be designated an appropriate Reliability Classification as detailed in Chapter 1. Also, the designer shall refer to the Wastewater Discharge Checklist, Appendix 1-A.

15.5 Influent Problems

15.5.1 General

Small treatment facilities are more sensitive to influent problems due to a reduction in hydraulic or organic buffering capacity. Small plants are much more susceptible to peak flow variations and nighttime, weekend or seasonal variations.

15.5.2 Characterization of the Waste and Flow

An accurate characterization of the waste and flow conditions must be projected for the site and must include FLOW, BOD5, AMMONIA AND GREASE. While best engineering judgments for waste characterizations are sometimes necessary, an attempt should be made to project this character from similar facilities, instead of the absolute use of flow tables. For example, excess ammonia should be considered during design of a treatment system for a rest stop, truck stop or recreational vehicle park. These types of facilities can have five times the normal influent ammonia of domestic systems.

15.5.3 Flow Equalization

Flow equalization shall be considered for all mechanical treatment plants whose variations in hydraulic and/or organic loadings might interfere with operations. Flow equalization shall be located after preliminary treatment facilities. Refer to Chapter 4 for specific design consideration.

15.5.4 Preliminary Treatment

Preliminary treatment involves the removal of large solids that could damage pumps and equipment in the downstream treatment process. Properly designed septic tanks (See Appendix 15-A) or, for mechanical plants, screening and grit removal, may be required. In the case of package activated sludge plants, comminutors are discouraged due to their tendency to create rope-like conditions which clog unit processes and pumps. A series of graded bar screens or rag catchers is preferable. A bypass channel utilizing a bar screen is required.

15.5.5 Grease and Oil

Restaurants shall be equipped with an effective grease and oil separator. Other potential sites of grease/oil production should be investigated by the design engineer. The sizing of the grease trap procedure will be forthcoming.

15.5.5.1 Grease and Oil Separator Design

One or more separators in series are required where grease or oil waste is produced that could hinder sewage disposal or treatment, and/or create line stoppages. Separators must be located so as to provide easy access for inspection, cleaning and maintenance. The dishwasher must not be connected to the grease trap in restaurants.

As vegetable oil usage has become more common, it should be understood that oils will not solidify until approximately 70 ^{o}F. or less. Therefore, the minimum design shall be a baffled, three-compartment, elongated chamber to allow for cooling. The minimum size of the separator shall be 1,500 gallons. The tank shall be buried, with manhole accesses to all compartments. Cleaning should be performed as required but no less than every three months.

15.6 Plant Operation and Certification

All wastewater treatment plants are required to be operated by certified operators. Copies of the Certification Regulations may be obtained from the Murfreesboro Fleming Training Center. The costs of operators' salaries and lack of reliable automation restrict good operation of package activated sludge plants. Passive systems will require less operator attention; hence, lower salary requirements.

15.7 Alternative Systems Design

The following systems are considered to be the predominant choices for small flow designs. Each system is suitable for a variety of applications and should be chosen based on Section 15.3.

15.7.1 Recirculating Sand Filter (RSF)

The RSF consists of a septic tank (See Appendix 15-A), recirculation tank, open bed sand filter with a special distribution system, and flow splitter device. The RSF treatment offers an economical alternative to the intermittent subsurface sand filter from which it evolved, and shares its characteristics of a quality effluent and simplicity of operations and maintenance. RSF designs are economically feasible between 1,000 to approximately 30,000 gallons per day. WHERE THE STEP SYSTEMS UTILIZE TURBINE PUMPS AND SCREENS IN THE WATERTIGHT TANKS, THE FLOWS MAY RUN FROM 1,000 TO AS MUCH AS 150,000 GALLONS.

15.7.1.1 General

The treatment system utilizes the following basic components:

a. Grease/oil separator(s), where needed
b. Watertight septic tank (See Appendix 15-A)
c. Recirculation tank with pump and filter
d. Open bed sand filter media to treat the wastewater
e. Distribution and collection systems to load the sand filter evenly
f. Flow Splitter
g. Disinfection of effluent

15.7.1.2 Hydraulic Flow

Submit the justification for design flow, both average and peak.

15.7.1.3 Septic Tank

A WATERTIGHT TANK IS REQUIRED. IT IS ALSO REQUIRED THAT SEPTIC TANKS (FOR A STEP SYSTEM) USE A TURBINE TYPE WATER WELL PUMP WITH A FILTER SCREEN. IT HAS BEEN SHOWN THAT THE PUMPING OF THE STEP TANKS MAY GO AS LONG AS FIFTEEN (15) YEARS BETWEEN PUMPINGS. GREASE/OIL TANKS MUST COMPARTMENTALIZED REQUIRING PUMPINGS AT LEAST EVERY THREE (3) MONTHS. THE ORENCO CONCEPT SEPTIC TANK WITH FILTER DOES NOT NEED TO BE COMPARTMENTALIZED.

15.7.1.4 GREASE - OIL SEPARATOR(S)

The removal of grease and oil in this system is very important. (SEE APPENDIX 15-B) Grease and oil separators should be oversized to give adequate detention time for cooling of the wastewater, particularly where automatic dishwashing is used. Maintenance of the separator should be scheduled. THE MINIMUM SIZE TANK OR SEPARATOR SHOULD BE 1,500 GALLONS CAPACITY. UNDER HEAVY FLOW CONDITIONS TANKS SHOULD BE IN SERIES TO ALLOW COOLING OF THE GREASE AND OIL TO AID COAGULATION OF THE OILS USED IN COOKING.

15.7.1.5 Recirculation Tank and Pump System

The tank serves as a wetwell for the septic tank EFFLUENT and filtered recirculated effluent to be pumped to the sand filter. The <u>effective</u> volume shall be EQUAL TO one days average flow. The tank shall be equivalent in strength and materials. No internal baffles are necessary. An access manhole is necessary for replacement of submersible dosing pumps if such are used.

Two alternating recirculation pumps are required. EACH RECIRCULATING PUMP MUST BE CONTROLLED BY A TIMER, IN CONTINUOUS CYCLES OF 5 MINUTES ON, 25 MINUTES OFF. THIS DOSING SCHEDULE PROVIDES 48 DOSING PERIODS IN A 24 HOUR PERIOD, ALLOWING THE INFLUENT/FILTRATE MIXTURE TO CYCLE THROUGH THE FILTER ABOUT 5 TIMES BEFORE DISCHARGE. FLOAT SWITCHES ARE WIRED IN PARALLEL WITH THE TIMER TO CONTROL THE PUMPS DURING PERIODS OF EXCESSIVE WASTEWATER FLOWS, AND INFLUENT OF TIMER MALFUNCTION. BOTH TIMER AND FLOAT SWITCH CONTROLS ARE REQUIRED.

A quick disconnect coupler and hanger pipe are recommended for pump removal and convenience.

15.7.1.6 Sand Filter Bed

The filter bed should be sized on the basis of 3.0-5.0 gallons per square foot per day of average strength domestic sewage (Carbonaceous BOD_5 of approximately 200 and influent ammonia of 15.).
The sand filter medium shall consist of 24-30 inches of clean coarse sand (chemically nonreactive) which has an effective size of 1.0 - 3.0 millimeters,

and a uniformity coefficient of less than 3.5. It shall be washed and free of clay and silt. Synthetic manufactured medium may be used which meets the criteria.

The bedding material supporting the filter sand shall consist of 6 inches of 1/4-inch to 1/2-inch stone. Below this layer SHALL BE 6 inches of a 1/2-inch to 1-inch stone. Below this SHALL BE 12 inches of 1-inch stone containing the underdrains. Two inches of compacted chokestone shall bed the 12-inch bottom layer. All support media shall be reasonably well graded with a low ratio of fines.

An impermeable plastic liner 20 mils thick is required for the bottom of the sand filter. The plastic liner may lie directly on the graded soil. The liner shall be properly seamed to form a leakproof basin for the filter and should be protected from puncturing. FENCING IS REQUIRED AROUND THE FILTER AND AROUND THE RECIRCULATION TANK.

15.7.1.7 Distribution and Collection Systems

The distribution system must be level. Distribution pipes shall be no smaller than 1 1/4-inches with appropriate holes drilled on site with sizing and spacing per the Low Pressure Pipe Design procedure available from the Division of Ground Water Protection. The holes shall be at the "4 o'clock and 8 o'clock" positions, 120^0 apart. Distribution pipes may rest upon concrete blocks. Splash plates need not be provided. Clean-out caps shall be provided on the ends of the distribution pipes.

15.7.1.8 Disinfection

See Chapter 10. THE USE OF ULTRAVIOLET DISINFECTION IS ENCOURAGED ON THE RSF AS ITS CLEAR EFFLUENT AND CONSTANT DISCHARGE WORKS WELL.

15.7.1.9 Flow Splitter

The flow splitter shall be designed so that recirculation rates can be easily controlled between a 1:1 and 5:1 recirculation ratio. A 4 to 1 ratio is the design recirculation ratio. AN OVERSIZED SPLITTER BOX IS RECOMMENDED

15.7.2 Artificial Wetlands

Artificial wetlands is an engineered marsh-like area which uses the physical, chemical, and biological processes in nature to treat wastewater, instead of using complicated mechanical systems. In the wetlands, organisms and plants use organics and nutrients in the wastewater for food. The pollutants are transformed into basic elements, plant biomass and compost. Several variations of wetlands have been developed including a marsh-pond-meadow, subsurface flow marsh or root-zone system, and a gravel marsh system.

The wetland may be used for total treatment (with appropriate preliminary screening) or as a polishing or tertiary addition to other processes.

Information on wetlands may be obtained from the Tennessee Valley Authority, Water Quality Branch, 270 Haney Building, Chattanooga, Tennessee 37401.

15.7.3 Lagoons and Hydrograph Controlled Release (HCR) Systems

See Chapter 9 for design details. A PRIMARY FLOW DEVICE OR CONTRACT WITH USGS TO DEVELOP A STREAM RATING CURVE MUST BE PROVIDED.

15.7.4 Aerobic Bio Reactors

Several manufacturers market aerobic bioreactors for various uses. Their use in domestic small flow applications is not yet widespread but appears promising.

Supplemental treatment will be required for ammonia removal and dissolved oxygen considerations. In addition, a polishing step may be required for BOD removal depending on permit requirements.

15.7.5 Package Activated Sludge Plants

Package Activated Sludge Plants will only be approved for design flows of 30,000 gpd or greater, after all other alternatives have been exhausted.

Among the various processes, the one most widely used for small treatment systems is the extended aeration process.

However, for any activated sludge or fixed film process, the criteria presented in Chapters 4, 5, 6, 7, 8, 10, 11, and 12 must be utilized for each unit process.

Of particular importance is the sludge production and wasting facilities. The design must include aerobic digestion or sludge holding for sludge wasting.

A sludge wasting schedule should be included in the engineering report to better define operator time requirements. The disposal site or landfill must be given. Where tertiary filters are employed, the use of an equalization tank is mandatory. Also, based on the Reliability Classification as determined by the appropriate WPC field office, multiple units and standby power (or a generator) may be required. These costs must be included in the cost effective/reliability analysis.

APPENDIX - A CHAPTER 15

SUGGESTED SPECIFICATIONS

A. ON-SITE INTERCEPTOR TANKS

1. GENERAL

   a. Interceptor tanks shall be modified 1000-gallon precast concrete, fiberglass or ABS and shall have been designed by a registered engineered and approved by the local regulatory agencies. The manufacturer shall provide the structural design and certification to the engineer for review. The design shall be in accordance with accepted engineering practice.

   b. The tanks shall be designed for the following loads:

   Top-300 psf Lateral Loads-62.4 psf

   Cold weather installations requiring deep burial will need special consideration.

   c. All tanks shall be guarantees in writing by the tank manufacturer for a period of two years from the date of delivery to the project. Manufacturer's signed guarantee shall accompany bids.

   d. Tanks shall be manufactured and furnished with access openings 18 inches in diameter and of the configuration shown on the drawings. Modifications of completed tanks will not be permitted.

   e. Inlet plumbing shall penetrate 18 inches into the liquid from the inlet flow line.

   f. Tanks shall be capable of successfully withstanding an above-ground static hydraulic test and shall be individually tested.

   g. All tanks shall be installed in strict accordance with the manufacturer's recommended installation instructions.

2. CONCRETE

   a. Walls, bottom and top of reinforced-concrete tanks shall be designed across the shortest dimension using one-way analysis. Stresses in each face of monolithically-constructed tanks may be determined by analyzing the tank cross-section as a continuos fixed frame.

   b. The walls and bottom slab shall be poured monolithically.

   c. Reinforcing steel shall be ASTM A-615 Grade 60, fy = 60,000 psi. Details and placement shall be in accordance with ACI 315 and ACI 318.

   d. Concrete shall be ready-mix with cement conforming to ASTM C150, Type II. It shall have a cement content not less than six (6) sacks per cubic yard and maximum aggregate size of 3/4 inch. Water/cement ratio shall be kept low (0.35+/-), and concrete shall achieve a minimum compressive strength of 5000 psi in 28 days.

   e. Tanks shall be protected by applying a heavy cement-base waterproof coating (Thoroseal or equal), on both inside and outside surfaces, in compliance with Council of American Building Officials (CABO) report # NRB-168;6181.

   f. Form release used on tank molds shall be Nox-Crete or equal. Diesel or other petroleum based products are not acceptable.

g. Tanks shall not be moved from the manufacturing site until the tank has cured for seven (7) days or has reached two-thirds of the design strength.

h. Tanks shall have a 1/2 inch wide by 1/2 inch deep groove, 21 inches, 24 inches or 30 inches in diameter, as required, surrounding the access opening. The groove shall be formed in the top of the tank at the time of manufacture to facilitate the installation of the riser.

i. In order to demonstrate watertightness, tanks shall be tested twice prior to acceptance. Each tank shall be tested at the factory, prior to shipping, by filling to the soffit and letting stand. After 24 hours, the tank shall be refilled to the soffit and the exfiltration rate shall be determined by measuring the water loss during the next two hours. Any leakage shall be cause for rejection. After installation is completed, each tank shall be filled with water and retested as previously described. If filled to the top of the riser, backfill of a depth equal to the height of the riser must be placed over the tank to prevent damage due to hydrostatic uplift.

B. RISERS & LIDS

1. INLET RISERS (required only on 2-compartment tanks and tanks with greater than 1500-gallon capacity) shall be ribbed PVC as manufactured by ORENCO SYSTEMS, INC., or equivalent. Risers shall extend to the ground surface and shall have a minimum nominal diameter of 21 inches.

2. OUTLET RISERS shall be ribbed PVC as manufactured by ORENCO SYSTEMS, INC., or equivalent. Risers shall be at least 12 inches high, shall have a minimum of nominal diameter of 24 inches when used with 12-inch or 15-inch diameter pump vaults, or 30-inch when used in a duplex application and shall be factory-equipped with the following:

a. Rubber Grommets. Two-1-inch diameter grommets, one for the splice box and one for the pump discharge, installed as shown on the drawing.

b. Adhesive. Two-part epoxy, one pint per riser, for bonding riser to tank. One quarter for 30-inch diameter.

3. LIDS shall be furnished with each riser. Lids shall be ORENCO SYSTEMS Model FL-21g, FL-24g, or FL-30g, or equivalent as appropriate, fiberglass with green non skid finish, and provided with elastomeric gasket, stainless steel bolts, and wench. The riser and lid combination shall be able to support a 2500 lb. wheel load. (Note: this is not to imply that PVC risers are intended for traffic areas. Please refer to section on traffic protection.)

4. INSULATION (If Required) Ridge closed-cell foam insulation of 2" or 4" thickness shall be bonded to the underside of the lid.

5. RISER INSTALLATION shall be accomplished according to the manufacture's instructions.

C. STEG - GRAVITY ASSEMBLIES

1. OUTLET RISER shall be ribbed PVC as manufactured by ORENCO SYSTEMS, INC., or equivalent. Risers shall extend to the ground surface, shall have a minimum nominal diameter ofd 21 inches. Two-part epoxy, one pint per riser, for bonding riser to tank.

2. LID shall be furnished with each riser. Lids shall be ORENCO SYSTEMS Model FL-21g, or equivalent, fiberglass with green non skid finish, and provided with elastomeric gasket, stainless steel bolts, and wench. The riser and lid combination shall be able to support a 2500 lb. wheel load. (Note: this is not to imply that PVC risers are intended for traffic areas. Please refer to section on traffic protection.)

3. RISER INSTALLATION shall be accomplished according to the manufacture's instructions.

4. EFFLUENT FILTER Gravity system tanks for single-family dwellings shall be equipped with the ORENCO SYSTEM Model F-1248125 Effluent Filter, or equivalent, installed in conformance with the standard plans. (Note: Commercial and multiple-user tanks require larger Effluent Filters, the sizes of which must be individually determined and spelled our in the specifications.) The Effluent Filter shall consist of a 12-inch diameter, 48-inch deep PVC vault with eight (8) 1-1/4-inch diameter holes evenly spaced around the perimeter, 16-inches up from the bottom, and with a fiberglass base. Housed inside the PVC vault shall be a 1/8-inch mesh polyethylene screen. The 1-1/4-inch diameter vertical intake pipe within the screened vault shall have an overflow protection screen on the top and a one 1/2-inch diameter hole near the base for flow modulation. The Effluent Filter shall also be equipped with 5-1/2 feet of 1-1/4-inch flexible PVC flex hose with a plastic quick-disconnect fitting on the vault end. (For sites with riser greater than 24 inches in height, hose length shall be increased by one foot for each additional foot of riser. Also furnished shall be PVC flex hose bushed to fit a 4-inch sanitary tee, air relief vent and fittings as shown on the plans.) The Effluent Filter shall be suspended from the top of the septic tank by supports which shall be provided by ORENCO SYSTEMS, INC., or equivalent. The lateral from the tank to the collection line shall be laid to a uniform grade with no high points.

D. STEP PUMPING ASSEMBLIES for Single-Family Dwellings

1. MATERIALS All pumping systems shall be ORENCO SYSTEMS High-Head Pumping Assemblies or equivalent composed of:

a. Risers & Lids. Same as B., through 5, above.
b. Screened Pump Vault. Model SV1260Fi or SV1548Fi, PVC vault, or equal, fitted with 1/8-inch mesh polyethylene screen and a 4-inch diameter PVC flow inducer for a high head pump.
c. Discharge Hose and Valve Assembly. Model HV100BX, or equal, 1-inch diameter, 150 psi PVC ball valve, PVC flex hose with working pressure rating of 100 psi, Schedule 40 PVC pipe, and a 12-inch length of PVC flex hose with fittings to be installed outside the riser. Six-gpm flow controller optional.
d. Mercury Switch Float Assembly. Model MF-ABR, or equivalent, with three mercury switch floats mounted on a fixed PVC stem attached to the pump vault. The high- and low- level alarms and on-off functions shall be present as shown on the drawing. Each mercury switch float shall be secured with a nylon strain relief bushing. The "A" & "R" floats shall be UL- or CSA- listed and shall be rated for 4.5 A@ 120 V. The "B" float shall be UL- or CSA- listed and shall be rated for 13 A@ 120 V.
e. High-Head Effluent Pump. Model 8 OS105HH, or equal, 1/2 Hp, 115V, single phase, 60 Hz, 2-wire motor, 8-foot long extra heavy duty (SO) electrical cord with the ground to motor plug. Pump shall be UL listed as an effluent pump. (Note: if working heads over 150 feet are expected, a Model 8OS107HH or larger equivalent pump may be specified.)

f. Electrical Splice Box. UL approved for wet locations, equipped with four (4) electrical cord grips and a 3/4-inch outlet fitting. Also included shall be UL-listed butt splice connectors. (Note: Specifications for the EY conduit seal shall be covered in another section.)
g. Controls & Alarms. Model A-1RO, or equivalent control panel with the following:

1) Redundant-Off Relay: 115V., automatic, single pole.
2) Audible Alarm: Panel mount with a minimum of 80 dB sound pressure at 24 inches. Continuous sound.
3) Visual Alarm: NEMA 4-rated, 7/8-inch diameter, oiltight, with push-to-silence feature.
4) Audio-Alarm Reset Relay: 115 V, automatic, with DIN rail mount socket base

.

5) Toggle Switch: 15 amp motor rated, single -pole, double-throw with three positions: manual (MAN), (OFF) and automatic (AUTO).
6) Fuse Disconnect: DIN rail mount socket base with 2 amp SLO BLOW fuse.
7) Current-Limiting Circuit Breaker: Rated for 20 amps, OFF/ON switch, DIN rail mounting with thermal magnetic tripping characteristics.
8) Enclosure: NEMA 4X-rated, fiberglass with stainless steel or non-metallic hinges, stainless steel screws and padlockable latch. 8-inches high X 6-inches wide X 5-1/8 inches deep.
9) Alarm Circuit: Wired separately from the pump circuit so that, if the pump's internal overload switch or current-limiting circuit breaker is tripped, the alarm system remains functional.

2. INSTALLATION All pumping systems shall be installed in accordance with the manufacture's recommendations and the standard plans.
3. LOCATION The pump control panel shall be mounted on the side of the house nearest the tank and pump. NEC requires that the control panel be located within 50 feet of and within sight of the pump.

E. STEP PUMPING ASSEMBLIES for Commercial or Multiple-User Tanks

Note: Standard dimensions and materials for specifying pumping assemblies for other than single-family dwellings can be found in OSI's price list and in OSI's design-aid chart entitled "Commercial & Multiple User Effluent Pumping Systems". Engineers at OSI are available to provide assistance. The sizing of the larger capacity tanks should be based on a 4 to 1 length to width ratio.

CAUTION

To avoid accidents and limit liability, districts should issue frequent reminders to their constituents that:

1. Open manholes are potentially hazardous, so it is essential that the lids be bolted securely at all times.
2. The atmosphere in interceptor tanks can be dangerous, so maintenance should be performed only by trained personnel.
3. Control/alarm panels should be mounted our of the reach of small children and must be kept locked.

APPENDIX 15-B

Grease Disposal Tips

Suggestions For Preventing Grease Plugs

à Train employees not to pour grease down the drain.

à Wipe food off dirty dishes into the trash before you load the dishwasher.

à Recycle your grease. Several grease rendering companies are located in Tennessee. One of these can supply you with a grease barrel and pick up the grease when the barrel is full. The cost for this service is quite low.

à Install a pretreatment device. Grease traps, grease interceptors and grease devices can be effective in removing grease from washwater. These devices can range in size from a small truck to large underground tanks. They need regular maintenance. Depending on your volume of grease and the size of your system, you may need to check it twice a day or twice a year! If you decide to install a system, check with the Section 15.7.1.4 for sizing instructions.

à Use enzyme or bacterial additives to cut down on grease.   Many local vendors supply enzymes and bacteria which consume and convert it into harmless by-products. These products work best in conjunction with a grease trap or interceptor. Product success varies greatly. Take special precaution to find products that will not harm your septic system.

Things To Avoid Doing

* Pouring fats, oil or grease down the drain causes problems. When these materials meet cooler water in the sewer line, they can get hard and coat the sewer line. Eventually this could cause a blockage in the sewer line and sewage could back up into your business. (This is similar to cholesterol causing plaque deposits in your blood vessels and causing heart attacks.)

* If you pour grease down the drain with plenty of hot water, you are just washing the problem down stream. State parameters for oil and grease limit the amount of oil and grease in the wastewater to 30 parts per million. That's approximately one teaspoon in 43 gallons of water. That's not much at all. Violators are subject to penalties as described in the Tennessee Water Quality Control Act including fines of up to $10,000 dollars per day.

* If grease clogs your side sewers, you can end up with a sewage backup in your business. That's a serious problem that can cause the Health Department to temporarily close your business. If grease blocks the city's sewer lines, sewage can back up into many businesses and homes and you could be held liable.

28 March 1994

CHAPTER 16

Slow Rate Land Treatment

SLOW RATE LAND TREATMENT

16.1 General

16.1.1 General

This chapter provides guidelines and criteria for the design of slow rate land treatment systems. It is not applicable to overland flow or rapid infiltration.

There are basically two types of slow rate systems. Type 1 systems are designed to apply the maximum amount of wastewater to the minimum amount of land area. The wastewater loading rate is limited by the maximum amount of a particular wastewater constituent that can be applied to a specific site. For wastewater from municipalities, the limiting design factor is usually either the hydraulic capacity of the soil or the nitrogen content of the wastewater. For industrial wastewater, the limiting design factor may be the hydraulic capacity of the soil, nitrogen or any other wastewater constituent such as metals, organics, etc. Type 2 systems are designed to apply the available wastewater to the maximum land area possible. The objective is usually crop irrigation and the design involves determining the water needs of the particular crop.

Although this chapter is written around Type 1 systems, the methodology can be adapted to satisfy Type 2 systems.

16.1.2 Applicability

Slow rate systems are designed and operated so that there is no direct discharge to surface waters. Disposal is by evaporation directly to the atmosphere, by transpiration to the atmosphere via vegetation uptake and by percolation to groundwater. A State of Tennessee Operating Permit is required for operation of slow rate land treatment systems.

16.1.3 Location

The disposal site should generally be relatively isolated, easily accessible and not susceptible to flooding. The site can be developed on agricultural land and/or forests or can include parks, golf courses, etc. Site location shall take into account dwellings, roads, streams, etc. A site approval by the Division will be required before review of the Engineering Report.

16.1.4 Topography

Maximum grades for wastewater spray fields should be limited to 8% for row crops, 15% for forage crops and 30% for forests. The maximum grade for any surface spreading system should be 10%. Ideally, any site should have a minimum slope of 2 - 3%. Sloping sites promote lateral subsurface drainage and make ponding and extended saturation of the soil less likely than on level sites. Depressions, sink holes, etc., are to be avoided.

16.1.5 Soils

In general, soils with a USDA Soil Conservation Service permeability classification of moderately slow (0.2 to 0.6 inches/hour) or more are suitable for wastewater irrigation. However, groundwater and drainage conditions must also be suitable. Soils which are poorly drained, have high groundwater tables or restrictive subsurface soil layers are not suitable for slow rate land treatment without drainage improvements.

16.2 Soil Investigations

16.2.1 General

The land treatment soil investigation must characterize the infiltration rate, permeability, and chemical properties of the first 5 to 10 feet of the soil profile. It must verify Soil Conservation Service soil mapping. It must also determine the elevation of the seasonal high groundwater, establish the groundwater flow direction and gradient, and identify any subsurface conditions which may limit the vertical or lateral drainage of the land treatment site. The number of soil samples necessary to supply all of this information will be dependent on the nature of the particular site. As a minimum, however, TDHE recommends that at least one sample be taken for every acre in order to develop a detailed soils map of the site for the Engineering Report. Samples from soils with similar characteristics can be combined and the analyses can be performed on each soil group sample.

16.2.2 Saturated Hydraulic Conductivity Testing

Saturated vertical hydraulic conductivity testing is required for the most limiting horizon of each soil series present. The most limiting soil horizon should be determined from soil survey information. A minimum of three (3) tests for each soil series should be performed, unless the flooding basin method is used, in which case, only one test per series is needed. Testing for saturated horizontal hydraulic conductivity is additionally required when subsurface drainage systems are planned or when lateral subsurface drainage is the predominant drainage mechanism for the land treatment site.

Acceptable methods for saturated hydraulic conductivity testing are listed in Table 16-1. Percolation tests as performed for septic tank drain fields are not acceptable.

16.2.3 Soil Chemical Testing

The pH, Cation Exchange Capacity, and Percent Base Saturation, of each soil series must be determined from samples taken from the A and B horizons. These chemical tests determine the retention of wastewater constituents in the soil and the suitability of the soil for different cover crops. A minimum of three (3) samples for each soil series should be taken. The samples can be mixed together and tested for each soil series if the series is uniform. Testing for soil nutrients (nitrogen, phosphorus and potassium) and agronomic trace elements may be included if appropriate for the vegetative management scheme.

Soil chemical testing should be in accordance with the latest edition of Methods of Soil Analysis published by the American Society of Agronomy, Madison, Wisconsin.

16.3 Preapplication Treatment Requirements

16.3.1 General

Wastewater irrigation systems have a demonstrated ability to treat high strength organic wastes to low levels. However, such systems require a high degree of management with particular attention paid to organic loading rates and aeration of the soil profile between wastewater applications.

The TDHE requires that all domestic and municipal wastewaters receive biological treatment prior to irrigation. This is necessary to:

a. Protect the health of persons contacting the irrigated wastewater.

b. Reduce the potential for odors in storage and irrigation.

Some industrial wastewaters may be suitable for direct land treatment by irrigation under intensive management schemes. The TDHE will evaluate such systems on a case-by-case basis.

16.3.2 BOD and TSS Reduction, and Disinfection

Preapplication treatment standards for domestic and municipal wastewaters prior to storage and/or irrigation are as follows:

a. Sites Closed to Public Access

All wastewater must be treated to a level afforded by lagoons which are designed in accordance with chapter 9.

Disinfection is generally not required for restricted access land treatment sites. The TDHE may, however, require disinfection when deemed necessary.

b. Sites Open to Public Access

Sites open to public access include golf courses, cemeteries, green areas, parks, and other public or private land where public use occurs or is expected to occur. Wastewater irrigated on public access sites must not exceed a 5-day Biochemical Oxygen Demand and Total Suspended Solids of 30 mg/l, as a monthly average. Disinfection to reduce fecal coliform bacteria to 200 colonies/100 ml is required.

The preapplication treatment standards for wastewater that is to be applied to public access areas will be reviewed by the TDHE on a case-by-case basis. More stringent preapplication treatment standards may be required as the TDHE deems necessary. TDHE recommends that the engineer give preference to pretreatment systems that will provide the greatest degree of reliability.

16.3.3 Nitrogen

Maximum nitrogen removal occurs when nitrogen is applied in the ammonia or organic form. Nitrate is not retained by the soil and leaches to the groundwater, especially during periods of dormant plant growth. Therefore, the preapplication treatment system must <u>not</u> produce a nitrified effluent.

The TDHE recommends that aerated or facultative wastewater stabilization ponds be used for preapplication treatment where possible. These systems generally produce a poorly nitrified effluent well-suited for wastewater irrigation. When mechanical plants are employed for preapplication treatment, they should be designed and operated to limit nitrification.

The Engineering Report should indicate the expected range of nitrogen removal in the preapplication treatment system. Predictive equations for nitrogen removal in facultative wastewater stabilization ponds have been developed by Pano and Middlebrooks (1982), and Reed (1985).

16.3.4 Treatment and Storage Ponds

The storage pond and irrigation pump station must be hydraulically separate from the treatment cells (i.e., pumping must not affect hydraulic detention time in these cells). The TDHE recommends the use of Chapter 9 of the Design Criteria for Sewage Works, as well as the United States Environmental Protection Agency's October 1983 Design Manual: Municipal Wastewater Stabilization Ponds as a reference for design of preapplication treatment ponds.

16.4 Inorganic Constituents of Treated Wastewater

Inorganic constituents of effluent from preapplication treatment should be compared with Table 16-2 to insure compatibility with land treatment site soils and cover crops.

16.5 Protection of Irrigation Equipment

Prior to pumping to the spray field distribution system, the wastewater must be screened to remove fibers, coarse solids, oil and grease which might clog distribution pipes or spray nozzles. As a minimum, screens with a nominal diameter smaller than the smallest flow opening in the distribution system should be provided. Screening to remove solids greater than one third (1/3) the diameter of the smallest sprinkler nozzle is recommended by some sprinkler manufacturers. The planned method for disposal of the screenings must be provided.

Pressurized, clean water for backwashing screens should be provided. This backwash may be manual or automated. Backwashed screenings should be captured and removed for disposal. These screenings should not be returned to the storage pond(s) or preapplication treatment system.

16.6 Determination of Design Percolation Rates

16.6.1 General

One of the first steps in the design of a slow rate land treatment system is to develop a "design percolation rate" (Perc). This value is used in water balance calculations to determine design wastewater loading(s) and, thus, spray field area requirements. The percolation rate is a function of soil permeability and drainage.

16.6.2 Design Values

The most limiting layer; i.e., A, B or C horizon, of each soil series must be identified. Any surface conditions which limit the vertical or lateral drainage of the soil profile must also be identified. Examples of such conditions are shallow bedrock, a high water table, aquitards, and extremely anisotropic soil permeability.
Values of saturated vertical hydraulic conductivity from soil testing are used to develop the design percolation rate.

Values of saturated vertical hydraulic conductivity must be modified by an appropriate safety factor to determine design percolation. The safety factor reflects the influence of several elements including: the fact that long periods of saturation are undesirable, the uncertainty of test values, the drainage characteristics of the land treatment site, the variation of permeability within and between different soil series, the rooting habits of the vegetation, the soil reaeration factors, and the long-term changes in soil permeability due to wastewater application. The TDHE recommends that the design percolation rate of land treatment sites be no more than 10 percent of the mean saturated vertical hydraulic conductivity of the most limiting layer within the first five feet from the surface, in accordance with the following equation:

$$\text{Perc} = K \times 0.10 \qquad \text{Eq. 16-1}$$

Where, Perc = Design percolation rate, (in/month)

K = Permeability of limitinng soil layer, (in/month)

0.10 = Safety factor

Sites with seasonal high groundwater less than 5 feet deep may require drainage improvements before they can be utilized for slow rate land treatment. The design percolation at such sites is a function of the design of the drainage system.

16.7 Determination of Design Wastewater Loading

16.7.1 General

The design wastewater loading is a function of:

a. Precipitation.
b. Evapotranspiration.
c. Design percolation rate.
d. Nitrogen loading limitations.
e. Other constituent loading limitations.
f. Groundwater and drainage conditions.
g. Average and peak design wastewater flows.

Therefore, developing the design wastewater loading is an iterative process. An initial value is selected from water balance calculations and used to determine wetted field area. This loading is then compared to nitrogen and other constituent loading limitations (reference Section 16.8). If the initial value exceeds these limitations, the design wastewater loading is reduced and the process is repeated. This iterative process is illustrated in Appendix 16-A.

16.7.2 Water Balance

Maximum allowable monthly wastewater hydraulic loadings are determined from the following water balance equation:

$$\text{Lwh} = (\text{PET} + \text{Perc}) - \text{Pr} \qquad \text{Eq. 16-2}$$

Where, Lwh = Maximum allowable hydraulic wastewater loading (in/month).

PET = Potential Evapotranspiration, (in/month)

Perc = Design percolation rate (in/month);

Pr = Five-year return monthly precipitation, (in/month).

Example water balance calculations are presented in Appendix A. From these, critical water balance months; i.e., months with the smallest allowable hydraulic wastewater loading, are identified.

16.7.3 Potential Evapotranspiration (PET)

Reliable field data for evapotranspiration are difficult to obtain. Therefore, values for average monthly potential evapotranspiration (PET) generated from vegetative, soil and climatological data are used in water balance calculations. The method used to estimate average monthly potential evapotranspiration for water balance calculations must be referenced in the Engineering Report. In addition, these values must be based on a record of 30 years of historical climatic data.

The Thornthwaite method is an empirical equation developed from correlations of mean monthly air temperatures with evapotranspiration from water balance studies in valleys of the east-central United States where soil moisture conditions do not limit evapotranspiration (The Irrigation Association, 1983, pp. 112 to 114). The Thornthwaite method is applicable to slow rate land treatment systems in the southeast United States, including Tennessee.

A modified version of the Thornthwaite equation is outlined below. Note that the results are expressed in inches, for a month period. Finally, for water balance calculations as described in this Section, a 30-year record of historical climatic data (referred to as the climatological normal) is required to determine monthly temperature normals used in the Thornthwaite equation.

$$PET = 0.63 \times S \times \left(\frac{50 \times (T-32)}{9 \times I}\right)^{A} \qquad \text{Eq. 16-3}$$

Where, PET = 30 - day Thornthwaite Potential Evapotranspiration,(in)

S = Daylight hours, in units of 12 hours

T = Mean (normal) monthly air temperature, in degrees Fahrenheit

I = Annual heat index obtained by summing the 12 monthly heat indexes, i, where:

$$i = \left(\frac{T-32}{9}\right)^{1.514}$$

A = Power term derived from annual heat index, I, where:

$$A = 0.000000675(I)^3 - 0.0000771(I)^2 + 0.01792(I) + 0.49239$$

Climatic information more appropriate to any specific location in Tennessee can be used, but its use must be documented in the Design Report. Also, other methods of calculating the PET can be used, provided that the use of an alternative method has been given prior approval by the TDHE.

16.7.4 Five-Year Return Monthly Precipitation

The TDHE requires the use of five-year return, monthly precipitation values in calculating the water balance. These values can be determined by either of the following methods:

a. Use the five-year annual rainfall and apportion this amount to each month, using each month's average for a 30-year period.

b. Pr = Pr(Ave) + (0.85 x std. dev.) Eq. 16-4

where Pr(Ave) = average monthly precipitation from a 30- year historic record

std. dev. = standard deviation for same

Thirty-year records of precipitation (as well as temperature) are available for specific locations in Tennessee as well as for the four geographic divisions,
shown in Figure 16-1. Climatic information can be obtained from the National Oceanic and Atmospheric Administration (NOAA)in Asheville, North Carolina. The source of any data that are used in designing a slow-rate irrigation system must be referenced in the Design Report.

16.8 Nitrogen Loading and Crop Selection and Management

16.8.1 General

Nitrate concentration in percolate from wastewater irrigation systems must not exceed 10 mg/L. Percolate nitrate concentration is a function of nitrogen loading, cover crop, and management of vegetation and hydraulic loading. The design wastewater loading determined from water balance calculations must be checked against nitrogen loading limitations. If, for the selected cover crop and management scheme, the proposed wastewater loading results in estimated percolate nitrate concentrations exceeding 10 mg/l, either the loading must be reduced or a cover crop with a higher nitrogen uptake must be selected.

16.8.2 Nitrogen Loading

In some instances, the amount of wastewater that can be applied to a site may be limited by the amount of nitrogen in the wastewater. A particular site may be limited by the nitrogen content of the wastewater during certain months of the year and limited by the infiltration rate during the remainder of the year.

Equation 16-5 is used to calculate, on a monthly basis, the allowable hydraulic loading rate based on nitrogen limits:

$$Lwn = \frac{Cp\,(Pr-PET) + U(4.424)}{(1 - f)(Cn) - Cp} \qquad \text{Eq. 16-5}$$

Where: Lwn = allowable monthly hydraulic loading rate based on nitrogen limits, inches/month

Cp = nitrogen concentration in the percolating wastewater, mg/l. This will usually be 10mg/l

Pr = Five-year return monthly precipitation, inches/month

PET = potential evapotranspiration, inches/month

U = nitrogen uptake by crop, pounds/acre/month

Cn = nitrogen concentration in applied wastewater, mg/l (after losses in preapplication treatment)

f = fraction of applied nitrogen removed by denitrification and volatilization.

The values of Lwh and Lwn are compared for each month. The lesser of the two values, designated as Lwd, will be used in subsequent calculations to determine the amount of acreage needed.

The monthly values for nitrogen uptake by crops, U, can be derived by several methods:

1. Assume that the annual nitrogen uptake is distributed monthly in the same ratio as is the PET.

2. If data on nitrogen uptake versus time are available for the crops and climatic region specific to the project under design, then such information may be used.

Appendix A contains an example that illustrates the use of equations 16-2 and 16-5.

16.8.3 Cover Crop Selection and Management

Row crops may be irrigated with wastewater only when not intended for direct human consumption. Livestock must not be allowed on wet fields so that severe soil compaction and reduced soil infiltration rates can be avoided. Further, wet grazing conditions can also lead to animal hoof diseases. Pasture rotation should be practiced so that wastewater application can be commenced immediately after livestock has been removed. In general, a pasture area should not be grazed longer than 7 days. Typical regrowth periods between grazings range from 14 to 35 days. Depending on the period of regrowth provided, one to three water applications can be made during the regrowth period. At least 3 to 4 days drying time following an application should be allowed before livestock are returned to the pasture. Unmanaged, volunteer vegetation (i.e., weeds) is not an acceptable spray field cover. Disturbed areas in forest systems must be initially grassed and replanted for succession to forest.

Spray field cover crops require management and periodic harvesting to maintain optimum growth conditions assumed in design. Forage crops should be harvested and removed several times annually. Pine forest systems should be
harvested at 20 to 25 year intervals. Hardwood forest systems should be harvested at 40 to 60 years. It is recommended that whole tree harvesting be considered to maximize nutrient removal. However, wastewater loadings following the harvesting of forest systems must be reduced until the hydraulic capacity of the site is restored. Spray field area to allow for harvesting and the regeneration cycle should be considered in design.

While high in nitrogen and phosphorus, domestic and municipal wastewaters are usually deficient in potassium and trace elements needed for vigorous agronomic cover crop growth. High growth rate forage crops such as Alfalfa and Coastal Bermuda will require supplemental nutrient addition to maintain nitrogen uptake rates assumed in design. Industrial wastewaters considered for irrigation should be carefully evaluated for their plant nutrient value.

16.9 Land Area Requirements

16.9.1 General

The land area to which wastewater is applied is termed a "field". The total land requirement includes not only the field area, but also land for any preapplication treatment facilities, storage reservoir(s), buffer zone, administration/maintenance structures and access roads. Field and buffer zone requirements are addressed in this Section. Land area for storage reservoirs is discussed in Section 16.10. All other land requirements will be dictated by standard engineering practices and will not be addressed in this document.

16.9.2 Field Area Requirements

The area required for the field is determined by using the following equation:

$$A = \frac{(Q_y + V)C}{Lwd} \qquad \text{Eq. 16-6}$$

where

- A = field area, acres
- Q_y = Flow, MG per year
- V = net loss or gain in stored wastewater due to precipitation, evaporation and/or seepage at the storage reservoir, gallons per day
- Lwd = design hydraulic loading rate, in/year

$$C = \frac{1{,}000{,}000 \text{ gal}}{\text{MG}} \times \frac{\text{ft}^3}{7.48 \text{ gal}} \times \frac{12 \text{ in}}{\text{ft}} \times \frac{\text{acre}}{43{,}560 \text{ ft}^2} = 36.83$$

The first calculation of the field area must be made without considering the net gain or loss from the storage reservoir. After the storage reservoir area has been calculated, the value of V can be completed. The final field area is then recalculated to account for V. The Appendix includes the use of Equation 16-6.

16.9.3 Buffer Zone Requirements

The objectives of buffer zones around land treatment sites are to control public access, improve project aesthetics and, in case of spray irrigation, to minimize the transport of aerosols. Since development of off-site property adjacent to the treatment site may be uncontrolable, the buffer zone must be the primary means of separating the field area from off-site property. Table 16-3 gives minimum widths of buffer zones for varying site conditions:

Table 16-3
On-Site Buffer Zone Requirements

	Surface Spread	Sprinkler Systems (Edge of Impact Zone) Open Fields	Sprinkler Systems (Edge of Impact Zone) Forested
Site Boundaries	100 ft.	300 ft.	150 ft.
On-site streams, ponds and roads	50 ft.	150 ft.	75 ft.

16.10 Storage Requirements

16.10.1 General

The design of a land application system must take into account that wastewater application will be neither continuous nor constant. Provisions must be made for containing wastewater when conditions exist such that either wastewater cannot be applied or when the volume of wastewater to be applied exceeds the maximum application rate.

The storage requirement can be determined by either of two methods. The first method involves the use of water balance calculations and is illustrated in Appendix A. The second method involves the use of a computer program that was developed based upon an extensive NOAA study of climatic variations throughout the United States. The program entitled EPA-2 would probably be the most appropriate of the three programs available. For information on the use of the computer program, contact the National Climatic Center of NOAA at (704) 259-0448.

16.10.2 Estimation of Storage Requirements Using Water Balance Calculations

The actual wastewater that is available is compared to the actual amount that can be applied. Any excess wastewater must be stored. The actual wastewater volume must be converted to units of depth for that comparison. Equation 16-7 will be used:

$$Wp = \frac{Qm \times C}{Ap} \qquad \text{Eq. 16-7}$$

where

Wp = depth of wastewater, in inches

Qm = volume of wastewater for each month of the year, in million gallons

$$C = \frac{1{,}000{,}000\ \text{gd}}{\text{MG}} \times \frac{\text{ft}^3}{7.48\ \text{gal}} \times \frac{\text{acre}}{43{,}560\ \text{ft}^2} \times \frac{12\ \text{in}}{\text{ft}} = 36.83$$

Ap = field area, in acres

The months in which storage is required are cumulated to determine the maximum amount of total storage needed. The use of the method is illustrated in Appendix A.

The maximum storage amount in inches, over the field area, is converted to a volume, in cubic feet. A suitable depth is chosen and a storage basin surface area is calculated.

This storage basin will be affected by three factors: precipitation, evaporation and seepage. These three factors are determined and the result is V, which is then introduced back into equation 16-6. A new, final field area is calculated and a corresponding new storage volume is determined.

In Tennessee, the maximum seepage is 1/4 inch per day. This amount can be used unless the storage basin will be constructed so that a lesser seepage rate will result. In some cases, where an impervious liner will be constructed, the seepage rate will be zero.

16.11 Distribution System

16.11.1 General

The design of the distribution system is a critical aspect of the land application. The field area and the storage volume were derived with the assumption that wastewater would be evenly distributed. For high strength wastes or wastes with high suspended/settleable solids, sprinkler applications are preformed. Sprinklers will distribute these wastes more evenly over the treatment area whereas surface application may result in accumulation of solids and odors near the application point.

16.11.2 Surface Spreading

With surface spreading, wastewater is applied to the ground surface, usually by perforated pipe or by an irrigation-type ditch, and flows uniformly over the field by gravity. The uniform flow is critically dependent upon a constant slope of the field, both horizontal and perpendicular to the direction of flow. Several other factors are of importance:

a. Uniform distribution cannot be achieved on highly permeable soils. The wastewater will tend to percolate into the soil that is nearest to the point of application.

b. A relatively large amount of wastewater must be applied each time so that wastewater will reach all portions of the field. The dosing must account for the fact that the field area nearest the point of application will be wetted for a longer period of time and, thus, will percolate more wastewater.

c. Erosion and/or runoff may be a problem. Since a surface discharge will not be allowed to occur, a return system may be necessary.

16.11.3 Sprinkler Spreading

Sprinkler systems can be classified into one of three general categories: (1) solid set, (2) portable and (3) continuously moving. The following factors should be considered during design:

a. The hydraulic conditions within the distribution system must be given a thorough review. Head losses through pipes, bends, nozzles, etc., must be balanced so that the wastewater is uniformly applied to the field.

b. Design must consider the effects of cold weather. Nozzles, risers, supply pipes, etc., must be designed to prevent wastewater from freezing in the various parts.

c. Wind can distort the spray pattern. Also, aerosols may be carried off the field area. A properly designed buffer zone should alleviate most of the aerosol problems. Also, the O&M manual can include a provision which would prevent spraying when the wind velocity is high enough to carry wastewater off the field area.

d. Crop selection is important. The higher humidity level may lead to an increase in crop disease.

e. Higher slopes can be used than in surface spreading (see Section 16.1.3). Also, slopes do not need to be constant. Further, the type of crop is nearly unlimited. Forests can be irrigated with solid set sprinklers. Forage crops can be irrigated with any of the three basic types of systems.

f. The system layout must take into consideration the method that will be used for harvesting the crop.

16.12 Spray Irrigation of Wastewater from Gray Water Facilities

16.12.1 General

This Section provides criteria for facilities that produce a "gray water" wastewater. These facilities include coin-operated laundries, car washes and swimming pool backwash filters. Wastewater disposal requirements are not as complex as are those for domestic wastewater. An engineering report which provides information on the design of the facilities must be submitted to the Division.

16.12.2 Site Location

16.12.2.1 The Division of Water Pollution Control must inspect and approve the proposed site prior to any construction being undertaken.

16.12.2.2 The site must be chosen such that the operation of the system will not affect surrounding property owners. No surface runoff or stream discharge will be allowed.

16.12.3 Design Flow

Since these are service enterprises, the amount of wastewater that is generated is directly related to the desire of people to use the facilities. Thus, an estimate of the number of potential users (and frequency) is extremely important. Various factors must be taken into consideration:

a. A rural setting would tend to have a shorter daily usage period than would an urban location.

b. An area that is predominately single-family houses would tend to have a lesser usage rate for laundries and car washes than would an area with apartment complexes.

c. The amount of water that washing machines use will vary among manufacturers and models. The Division recommends the use of water-saving machines.

The engineer should use 250 gpd/washer for laundries and 700 gpd/bay for car washes unless more reliable data is available.

16.12.4 Pretreatment

16.12.4.1 General

Facilities that produce gray water have different pretreatment requirements, designed not only to the type of facility but also to the specific establishment.

16.12.4.2 Laundries

a. All laundry wastewater (does not include sanitary wastes) shall pass through a series of lint screens. A series will consist of five screens, starting with a screen with 1-inch mesh and ending with a screen that is basically equivalent to a window screen.

b. Since some detergents produce a wastewater with a pH in the range of 11.0 - 11.5, some type of pH adjustment may be necessary. This may occur as a retrofit if the vegetation in the spray plots is being stressed by the high pH.

c. Disinfection will generally not be required unless the operation of the facilities will result in a potential hazard to the public. The need for disinfection will be determined by the Division on a case-by-case basis.

16.12.4.3 Car Washes

a. All car wash wastewater shall pass through a grit removal unit. The flow-through velocity shall be less than 0.5 feet per second. The grit removal unit shall be constructed to facilitate the removal of grit.

b. The use of detergents with a neutral (or nearly neutral) pH is recommended. The use of high-pH detergents may require neutralization if the vegetation is being stressed by the high pH.

16.12.4.4 Swimming Pools

a. A holding tank/pond shall be provided to receive the backwash water from the swimming pool filters. The solids shall be allowed to settle to the bottom before the supernatant is removed for disposition on the spray plots.

b. Dechlorination may be required if the vegetation on the plots is being stressed by the chlorine in the water.

c. If the entire pool volume is to be emptied, by using the spray plots, the rate shall be controlled so as to not exceed the application rate that is specified in Section 16.7.1.

16.12.5 Field Requirements

16.12.5.1 The maximum wastewater that can be sprayed on a site is based either on the nitrogen content of the wastewater or an amount equal to 10% of the infiltration rate of the most restrictive layer of soil which shall be determined with input from a qualified soil scientist.

16.12.5.2 The application of wastewater shall alternate between at least two separate plots. Each plot shall not receive wastewater for more than three consecutive days and must have at least three days rest between applications. Reserve land area of equivalent capacity must be available for all greg water systems.

16.12.5.3 Ground slopes shall not exceed 30%. Extra precautions must be taken on steep slopes (15-30%) to prevent runoff and erosion.

16.12.5.4 The field shall be covered with a good lawn or pasture grass unless an existing forested area is chosen. The ground cover should be a sturdy perennial that will resist erosion and washout. Forested areas should be chosen so that installation of sprinkler equipment will not damage the root systems of the trees and will not produce runoff due to the usual lack of grass in forested areas.

16.12.6 Application Equipment

16.12.6.1 Sprinklers shall be of a type and number such that the wastewater will be evenly distributed over the entirety of a plot. Information on sprinklers shall be included in the engineering report. In forest plots, sprinklers shall be on risers which shall be tall enough to allow the wastewater to be sprayed above the undergrowth. Sprinklers shall be of the type that are not susceptible to clogging.

16.12.6.2 All piping (excluding risers) shall be buried to a depth that will prevent freezing in the lines. An exception to this burial requirement can be made in the case where piping will be laid in forested areas. Burial in this case may be difficult, expensive and may kill some trees. All risers shall be designed such that wastewater will drain from them when wastewater is not being pumped. This can be accomplished by either draining all lines back into the pump sump or by placing a gravel drain pit at the base of each riser. Each riser would necessarily be equipped with a weep hole. Particular attention must be given during the design so that the entire subsurface piping does not drain into these pits.

16.12.6.3 The engineering report must contain hydraulic calculations that show that each nozzle distributes an equivalent amount of wastewater. Differences in elevation and decreasing pipe sizes will be factors which need to be addressed.

16.12.6.4 The piping must be of a type that will withstand a pressure equal to or greater than 1-1/2 times the highest pressure point in the system. The risers should be of a type of material such that they can remain erect without support. The pipe joints should comply with the appropriate ASTM requirements. Adequate thrust blocks shall be installed as necessary.

16.12.6.5 A sump shall be provided into which the wastewater will flow for pumping to the spray plots. The pump can be either a submersible type, located in the sump, or a dry-well type, located immediately adjacent to the sump in a dry-well. The pump shall be capable of pumping the maximum flow that can be expected to enter the sump in any 10-minute period. The pump shall be operated by some type of float mechanism. The float mechanism shall activate the pump when the water level reaches 2/3 of the depth of the sump and should de-activate the pump before the water level drops to the point to where air can enter the intake.

If the distribution system is designed to drain back into the sump, the sump shall be enlarged to account for that volume.

If desired, the sump for laundries can also contain the lint screens. The screens shall, in any case, be constructed so that they cannot be bypassed. They shall be built so that they can be easily cleaned. A container shall be provided for disposal of the lint which is removed from the screens.

16.12.6.6 The pipe from the facility to the sump shall be large enough to handle the peak instantaneous flow that could be realistically generated by the facility. Flow quantities, head loss calculations, etc., shall be included in the engineering report.

16.12.7 Operation of System

16.12.7.1 The operator shall insure that wastewater is applied to alternate plots on a regular basis.

16.12.7.2 Monthly operating reports shall be submitted to the appropriate field office of the Division of Water Pollution Control. The parameters to be reported shall be delineated by field office personnel but should include, as a minimum, dates of spray plot alternation.

16.12.7.3 The owner of the system shall apply for and receive an operating permit from the Division prior to initiation of operation of the system.

16.12.7.4 The system operator shall inspect and maintain the pump and sprinklers in accordance with manufacturer's recommendations. An operations manual shall be located at the facility for ready reference.

16.12.7.5 The operator shall inspect the wastewater facilities on a regular basis. The inspection shall include the spray plots to determine whether or not runoff and/or erosion are or have occurred, the spray patterns of the sprinklers, the physical condition of the system (looking for damage due to adverse pH conditions), etc.

16.12.7.6 The spray plots shall be mowed on a regular basis to enhance evapotranspiration. Grass height shall not exceed 6".

16.12.7.7 The lint screen at laundries shall be cleaned on a schedule that is frequent enough to prevent upstream problems due to head loss through the screens. Disposition of the lint shall be in accordance with applicable requirements.

16.12.7.8 The grit traps at car washes shall be cleaned at a frequency that is sufficient to keep the trap in its designed operating condition.

16.12.7.9 If the car wash is equipped with an automatic wax cycle, the operator shall be especially attentive to the possibility of wax build-up on the sump, pump and all downstream piping.

16.12.7.10 The operator shall insure that the car wash facility is not used as a sanitary dumping station for motor homes or for washing trucks/trailers that are used for hauling livestock. If necessary, the facility shall be posted with signs which clearly indicate this prohibition.

16.12.7.11 The sludge holding tank/pond at a swimming pool facility shall be cleaned at a frequency that is sufficient to prevent solids from being carried over into the pump sump. Cleaning shall be performed in a manner that will minimize re-suspending the solids and allowing them to enter the pump sump.

16.13 Plan of Operation and Management

A plan of Operation and Management is required before an Operating Permit can be issued. The Plan is written by the owner or the owner's engineer during construction of the slow rate land treatment system. Once accepted by the Division, the Plan becomes the operating and monitoring manual for the facility and is incorporated by reference into the Permit. This manual must be kept at the facility site and must be available for inspection by personnel from the Tennessee Department of Health and Environment.

This Plan should include, but not be limited to, the following information:

16.13.1 Introduction

a. System Description:

1. A narrative description and process design summary for the land treatment facility including the design wastewater flow, design wastewater characteristics, preapplication treatment system and spray fields.

2. A map of the land treatment facility showing the preapplication treatment system, storage pond(s), spray fields, buffer zones, roads, streams, drainage system discharges, monitoring wells, etc.

3. A map of force mains and pump stations tributary to the land treatment facility. Indicate their size and capacity.

4. A schematic and plan of the preapplication treatment sytem and storage pond(s) identifying all pumps, valves and process control points.

5. A schematic and plan of the irrigation distribution system identifying all pumps, valves, gauges, sprinklers, etc.

b. Discuss the design life of the facility and factors that may shorten its useful life. Include procedures or precautions which will compensate for these limitations.

c. A copy of facility's Tennessee Operating Permit.

16.13.2 Management and Staffing

a. Discuss management's responsibilities and duties.

b. Discuss staffing requirements and duties:

1. Describe the various job titles, number of positions, qualifications, experience, training, etc.

2. Define the work hours, duties and responsibilities of each staff member.

16.13.3 Facility Operation and Management

a. Preapplication Treatment System:

1. Describe how the sytem is to be operated.

2. Discuss process control.

3. Discuss maintenance schedules and procedures

b. Irrigation System Management:

1. Wastewater Application. Discuss how the following will be monitored and controlled. Include rate and loading limits.

    (a) Wastewater loading rate (inches/week)

    (b) Wastewater application rate (inches/hour)

    (c) Spray field application cycles

    (d) Organic, nitrogen and phosphorus loadings (lbs/acre per month, etc)

2. Discuss how the system is to be operated and maintained.

    (a) Storage pond(s)

    (b) Irrigation pump station(s)

    (c) Spray field force main(s) and laterals

3. Discuss start-up and shut-down procedures.

4. Discuss system maintenance.

    (a) Equipment inspection schedules

    (b) Equipment maintenance schedules

5. Discuss operating procedures for adverse conditions.

    (a) Wet weather

    (b) Freezing weather

    (c) Saturated Soil

    (d) Excessive winds

    (e) Electrical and mechanical malfunctions

6. Provide troubleshooting procedures for common or expected problems.

7. Discuss the operation and maintenance of back-up, stand-by and support equipment.

c. Vegetation Management:

1. Discuss how the selected cover crop is to be established, monitored and maintained.

2. Discuss cover crop cultivation procedures, harvesting schedules and uses.

3. Discuss buffer zone vegetative cover and its maintenance.

d. Drainage System (if applicable):

1. Discuss operation and maintenance of surface drainage and runoff control structures.

2. Discuss operation and maintenance of subsurface drainage systems.

16.13.4 Monitoring Program

a. Discuss sampling procedures, frequency, location and parameters for:

1. Preapplication treatment system.

2. Irrigation System:

(a) Storage pond(s)

(b) Groundwater monitoring wells

(c) Drainage system discharges (if applicable)

(d) Surface water (if applicable)

b. Discuss soil sampling and testing:

c. Discuss ambient conditions monitoring:

1. Rainfall

2. Wind speed

3. Soil moisture

d. Discuss the interpretation of monitoring results and facility operation:

1. Preapplication treatment system.

2. Spray fields.

3. Soils.

16.13.5 Records and Reports

a. Discuss maintenance records:

1. Preventive.

2. Corrective.

b. Monitoring reports and/or records should include:

1. Preapplication treatment system and storage pond(s).

   (a) Influent flow

   (b) Influent and effluent wastewater characteristics

2. Irrigation System.

   (a) Wastewater volume applied to spray fields.

   (b) Spray field scheduling.

   (c) Loading rates.

3. Groundwater Depth.

4. Drainage system discharge parameters (if applicable).

5. Surface water parameters (if applicable).

6. Soils data.

7. Rainfall and climatic data.

APPENDIX A

Due to the complexity of working with all of the variables that are inherent with land application systems, the most beneficial use of these criteria might be afforded by designing a slow-rate irrigation system for a hypothetical town in Tennessee. The following information is given:

Given: The town is in the Cumberland Plateau Section

The first step involves Equation 16-2, the water balance equation:

$$Lwh = (PET + Perc) - Pr \qquad \text{Eq. 16-2}$$

The Thornthwaite equation, Equation 16-3, will be used to derive the potential evapotranspiration (PET) term:

$$PET = 0.63 \times S \times \left(\frac{50 \times (T-32)}{9 \times I}\right)^{A} \qquad \text{Eq. 16-3}$$

The use of this equation requires that daylight hours at the particular latitude and the monthly air temperatures be used. Tennessee lies between latitudes of about 35^{o} and 36^{o} 40'. Since the latitudinal distance in Tennessee is not large, the daylight hours at the 36^{o} latitude will be adequate for any town in Tennessee. Table A-1 lists the average monthly daylight hours, in units of 12 hours, 36^{o} latitude.

Table A-1
Monthly Average Daylight Hours (S), in Units of 12 hours, for the 36^{o} Latitude

	36^0
January	0.84
February	0.91
March	1.00
April	1.09
May	1.17
June	1.21
July	1.19
August	1.12
September	1.04
October	0.94
November	0.86
December	0.81

The National Oceanic and Atmospheric Administration has published information on air temperature. A 30-year monthly average for the Cumberland Plateau Section, for the period of 1951 - 1980, will be used. Table A-2 is used to show the monthly daylight hours, air temperature and PET for this system.

Table A-2
Data Used, and Results Derived, for PET

	S at 36 Degree Latitude	Air Temp. Degrees Fahrenheit	PET, inches per month
January	0.84	35.6	0.10
February	0.91	38.6	0.27
March	1.00	46.9	0.97
April	1.09	57.3	2.30
May	1.17	64.7	3.59
June	1.21	71.6	4.90
July	1.19	75.0	5.44
August	1.12	74.3	5.00
September	1.04	68.8	3.79
October	0.94	57.3	1.98
November	0.86	46.7	0.82
December	0.81	39.1	0.27
		TOTAL =	29.43

Air temperature data for a specific location can be used, but its use must be documented by the NOAA. Also, other methods of calculating the PET can be used, provided that the use of an alternate method has been given prior approval by the TDHE.

Table A-3 is an indication of the Pr value in Eq. 16-2. Section 16.7.4 contains Equation 16-4 which is used in this case:

Pr = Pr (average) + (0.85 x std. dev.) Eq. 16-4

Table A-3
Five-Year Annual Rainfall, Using the 30-Year
Average Monthly Rainfall and Standard Deviation

	30-Year Average Rainfall, Inches	Standard Deviation	Pr Inches

January	5.46	2.54	7.62
February	4.83	2.22	6.72
March	6.45	2.82	8.85
April	4.95	1.93	6.59
May	4.75	1.62	6.13
June	4.32	1.41	5.52
July	5.06	2.10	6.85
August	3.60	1.33	4.73
September	4.10	1.69	5.54
October	3.08	1.63	4.47
November	4.39	2.02	6.11
December	5.43	2.49	7.55
TOTAL	56.42		76.68

An assumption is made that a site, with adequate acreage, has been selected, based on a site study. The following information is given:

Given: the most limiting soil layer has an infiltration rate of 0.3 inches/hour.

0.3 in/hr x 24 hr/day x 7 day/week x 0.10 = 5.04 in/week.

The 0.10 figure is the 10 percent design percolation limit.

Given: Wastewater can be applied in January only ten days, due to frozen soil, snow cover, etc.

Given: Wastewater can be applied in February and December on only 20 days.

Equation 16-2 can now be used to determine the maximum allowable monthly hydraulic wastewater loading, Lwh. Table A-4 illustrates the results:

Table A-4
Determination of Maximum Allowable Monthly
Hydraulic Wastewater Loading, D (allowed), Inches/Month

	(1) PET	(2) Pr	(1)-(2) (3)	Perc. (4)	Lwh (5) (3)+(4)
January	0.10	7.62	-7.52	7.20	0
February	0.27	6.72	-6.45	14.40	7.95
March	0.97	8.85	-7.88	22.32	14.44
April	2.30	6.59	-4.29	21.60	17.31
May	3.59	6.13	-2.54	22.32	19.78
June	4.90	5.52	-0.62	21.60	20.98
July	5.44	6.85	-1.41	22.32	20.91
August	5.00	4.73	0.27	22.32	22.59
September	3.79	5.54	-1.75	21.60	19.85
October	1.98	4.47	-2.49	22.32	19.83
November	0.82	6.11	-5.29	21.60	16.31
December	0.27	7.55	-7.28	14.40	7.12
TOTALS	29.43	76.68	-47.25	234.00	187.07

Based upon a maximum infiltration rate of 5.04 in/week, a water loss (PET), and a precipitation water gain, column 5 illustrates the maximum yearly and monthly hydraulic wastewater application rates. These rates will be used in the design of the system unless other limitations occur.

The most important of those other limitations is the percolate nitrogen concentration. If percolating water from a slow rate (SR) system will enter a potable ground water aquifer, then the system should be designed such that the concentration of nitrate nitrogen in the receiving ground water at the project boundary does not exceed 10 mg/l. Section 16.8.1 indicates that the nitrate concentration in the percolate must not exceed 10 mg/l. The approach to meeting this requirement involves estimating an allowable monthly hydraulic loading rate based on an annual nitrogen balance and comparing these monthly
rates to the monthly rates that are based on an application rate of 2.5 inches/week.

Equation 16-5 is used to determine monthly wastewater application rates based on a nitrate concentration of 10 mg/l.

$$Lwn = \frac{Cp\ (Pr - PET) + U\ (4.424)}{(1-f)\ (Cn) - Cp} \qquad \text{Eq. 16-5}$$

The following information is given:

Given: Cp = 10 mg/l
Given: Cn = 25 mg/l
Given: f = 25%
Given: U = 200 pounds/acre/year. This uptake is not constant; rather, the uptake is at a minimum in the cold months and is at a maximum in the warm months. Table A-5 indicates what percentage of U was allocated to each month.
Given: Pr and PET have been developed previously and have been included in Table A-5.

The monthly use of Equation 16-5 is illustrated in Table A-5. Also, this table includes a comparison of the monthly rates that were developed from the infiltration and the nitrogen bases.

Table A-5
Determination of Maximum Allowable Monthly Hydraulic
Wastewater Loading Based on Nitrogen Concentration
Comparison Between Infiltration and Nitrogen Loading Rates

	(2) Pr in.	(1) PET in.	U %	(6) lbs.	(7) Lwn in./mo.	(5) Lwh in./mo.	(8) Lwd in./mo.
January	7.62	0.10	1	2	9.61	0	0
February	6.72	0.27	2	4	9.39	7.95	7.95
March	8.85	0.97	4	8	13.05	14.44	13.05
April	6.59	2.30	8	16	12.99	17.31	12.99
May	6.13	3.59	12	24	15.04	19.78	15.04
June	5.52	4.90	15	30	15.88	20.98	15.88
July	6.85	5.44	17	34	18.80	20.91	18.80
August	4.73	5.00	15	30	14.86	22.59	14.86
September	5.54	3.79	12	24	14.13	19.85	14.13
October	4.47	1.98	8	16	10.94	19.83	10.94
November	6.11	0.82	4	8	10.09	16.31	10.09

December	7.55	0.27	2	4	10.34	7.12	0
TOTALS	76.68	29.43	100	200	155.12	187.07	133.73

As can be seen in Table A-5, soil infiltration is the limiting factor in the months of December, January and February. All other months have a limiting factor that is based on the nitrogen uptake rates of the crop.

The preliminary amount of land, Ap, that will be necessary for application of wastewater is determined by using Equation 16-6:

$$Ap = \frac{(Q_y + \ \ V)\ C}{(Lwd)} \qquad \text{Eq. 16-6}$$

The equation will be first solved without using the V term. The following information is given:

Given: Qy = MG per year = 36.5 MG
Given: Lwd = 133.73 inches/year (see column (8) Table A-5)
Given: C = 36.83

Substituting into Equation 16-6 gives the following:

Ap = 10.05 acres

This preliminary acreage is used in determining storage needs. When the storage requirements are determined, the V term can then be a derived and the actual field area, Af, can be calculated.

Storage volume requirements will be performed here by using water balance calculations. The basic steps are as follows:

1. The available monthly wastewater volume is converted to a unit of depth, in inches, by using the following equation:

$$Wp = \frac{Qm \times 36.83}{Ap} \qquad \text{Eq. 16-7}$$

In using the equation, the Qm term is assumed to be either 3.1 MGM, 3.0 MGM or 2.8 MGM, depending on the number of days in any particular month. No yearly variation is taken into account. In actuality, infiltration and inflow (I/I) and daily flow variations will require actual flow values.

Table A-6 is illustrative of the use of Eq. 16-7.

Table A-6
Estimation of Storage Volume Requirements
Using Water Balance Calculations

	(8) Lwd Cumulative	(9) Wp	(10) Change (9)-(8)	(11) Storage
January	0.00	11.36	11.36	24.04
February	7.95	10.26	2.31	26.35(b)
March	13.05	11.36	-1.69	24.66
April	12.99	10.99	-2.00 22.66	

May	15.04	11.36	-3.68	18.98	
June	15.88	10.99	-4.89	14.09	
July	18.80	11.36	-7.44	6.65	
August	14.86	11.36	-3.50	3.15	
September	14.13	10.99	-3.14	0.01(c)	
October	10.94	11.36	0.42(a)	0.42	
November	10.09	10.99		0.90	1.32
December	0.00	11.36		11.36	12.68
	133.73	133.74			

(a) Starting at October, in this example, will result in the maximum storage.
(b) Maximum storage.
(c) Rounding error; assume zero.

The storage volume is calculated by multiplying the maximum cumulative storage by the field area, as indicated below:

Storage volume = (26.35 in) (10.05 acres) (ft/12 in) (43,560 ft^2/acre)

= 961,000 ft^3 (rounded off)

The storage volume will be dependent upon three factors: precipitation, evaporation, and allowed seepage. To obtain the final volume, the following steps are used:

1. Calculate the area of the storage volume.

   Assume a maximum depth of 10 feet

   Area = Volume _ depth

   Area = 961,000 ft^3 _ 10 ft

   Area = 96,100 ft^2

2. Determine the monthly gain or loss in storage volume due to precipitation, evaporation and seepage in accordance with the following equation (see Table A-7):

   Vm = (Pr - evaporation - seepage)

   Column 14 is the result of using this equation. Precipitation has been presented previously in Table A-5. Evaporation is assumed to be 20 inches per year, distributed monthly in the same ratios of monthly PET to annual PET. Seepage rate shall not exceed 1/4 inch per day, in accordance with criteria in Chapter 9.

   Vm is converted from inches (Column 14) to MG (Column 15) by using the following equation:

   Vm = (Column 14) x 1 ft/12 in x 96,100 ft^2 x 7.48 gal/ft^3 x 1 MG/1,000,000 gal

   Vm = (Column 14) x 0.0599

3. The monthly storage losses and gains are added for a yearly total, Vt. This term is inserted back into Eq. 16-6 to calculate the actual, final field area.

   $$A = \frac{(Qy + Vt)C}{Lwd} \qquad \text{Eq. 16-6}$$

   where Qy = 36.5 MG

V_t = -2.073 (from Column 15, Table A-7)

C = 36.83

Lwd = 133.73 in/year

Substituting into Eq. 16-6 yields the following:

A_f = 9.48 acres

4. The water loss or gain is substracted or added to the monthly available wastewater, previously used in Eq. 16-7 (see Columns 15, 16 and 17, Table A-7).

5. The monthly available wastewater amounts, from column 17 of Table A-7, are converted to depths, in inches, by using Eq. 16-7.

$$W_f = \frac{Q_m \times (36.83)}{A_f} \quad \text{Eq. 16-7}$$

where

Q_m = MG

A_f = 9.48 acres

6. Substituting the monthly values of Q_{mf} into Eq. 16-7 yields column 18 of Table A-7. This is the amount of wastewater that will be available, in inches per month, for application to the field.

7. The available wastewater will be limited to field application due to weather, soil conditions, etc. This has been determined previously, was shown as Column 8 in Table A-5 and is re-indicated in Column 8 in Table A-7.

8. The difference between available wastewater and the amount that can be applied to the field is indicated in Column 19 of Table A-7. This column is derived by subtracting Column 8 from Column 18. A positive number indicates that more wastewater is available than can be applied; thus, storage is necessary. A negative number indicates that the soil can receive more wastewater than is received on a daily basis; thus, the wastewater that has been stored can be applied to the field along with the daily flow.

9. The cumulative storage is re-calculated, beginning with the storage basin(s) empty; in this case, at the beginning of October. This cumulative storage is shown in Column 20 of Table A-7 and indicates that a storage basin must be large enough to contain a volume of water equal to 27.00 inches of wastewater over the field area of 9.48 acres.

   The final storage volume is determined as follows:

   Vol. = (27.00 in) (9.48 acres) (ft/12 in) (43,560 ft^2/acre)

   Vol. = 929,000 ft^3 (rounded off)

10. Without changing the surface area of 96,100 ft^2, the depth is re-calculated:

    Depth = Volume _ area

    = 929,000 ft^3 _ 96,100 ft^2

Depth = 9.67 feet

//w//

Table A-7

	(2) Pr inches	(12) Evap. inches	(13) Seepage, inches	(14) Water loss/gain, V (2)-(12)-(13) inches	(15) Qm MG	(16) Wastewater Qmf MG	(17) Wf (16)+(15)
January	7.62	0.07	7.75	-0.20-0.012	3.1	3.088	12.00
February	6.72	0.18	7.00	-0.46	-0.028	2.8	2.772
March	8.85	0.66	7.75	0.44 0.026	3.1	3.126	12.14
April	6.59	1.56	7.50	-2.47	-0.148	3.0	2.852
May	6.13	2.44	7.75	-4.06	-0.243	3.1	2.857
June	5.52	3.33	7.50	-5.31	-0.318	3.0	2.682
July	6.85	3.70	7.75	-4.60	-0.276	3.1	2.824
August	4.73	3.40	7.75	-6.42	-0.385	3.1	2.715
September	5.54	2.58	7.50	-4.54	-0.272	3.0	2.728
October	4.47	1.34	7.75	-4.62	-0.277	3.1	2.823
November	6.11	0.56	7.50	-1.95	-0.117	3.0	2.883
December	7.55	0.18	7.75	-0.38	-0.023	3.1	3.077
Total 76.68	20.00	91.25	-34.57-2.073	36.5	34.427	133.75	133.73

RDL/E6078048
Appendix A
Sewer Regs

Table 16-1
HYDRAULIC CONDUCTIVITY TEST METHODS

1.0 SATURATED VERTICAL HYDRAULIC CONDUCTIVITY[a]

1.1 Laboratory Tests:[b]

Constant Head Method (coarse grained soils)	ASTM D 2434-68 AASTHO T 215-70 Bowles (1978), pp 97-104 Kezdi (1980), pp 96-102
Falling Head Method[c] (cohesive soils)	Bowles (1978), pp 105-110 Kezdi (1980), pp 102-108

1.2 Field Tests:

Flooding Basin Method[c]	U.S. EPA (1981), pp 3-13 to 15
Ring Permeameter Method	Boersma (1965) U.S. EPA (1981), pp 3-22 to 23
Double Tube Method[c]	Bouwer and Rice (1967) U.S. EPA (1981), pp 3-17 to 24
Air-Entry Permeameter[c] Method	Bouwer (1966) Reed and Crites (1984), pp 176 to 180 Topp and Binns (1976) U.S. EPA (1981), pp 3-24 to 27

2.0 SATURATED HORIZONTAL HYDRAULIC CONDUCTIVITY[d]

2.1 Field Tests:

Auger Hole Method	Reed and Crites (1984), pp 165 to 168 U.S. EPA (1984), pp 3-32 to 35 U.S. Dept. of Interior (1978), pp 55-67
Slug Test	Bouwer and Rice (1976)

a Other methods, properly documented, may be accepted by the TDHE. However, "standard" percolation tests as performed for septic tank drain fields are not acceptable.

b These tests require undisturbed field samples properly prepared to insure saturation. Reconstructed field samples are not acceptable. A description of the field sampling technique should accompany the laboratory testing results.

c Methods recommended by the TDHE.

d Testing for saturated horizontal hydraulic conductivity is required at land treatment sites where drainage improvements are planned and where lateral, as opposed to vertical, subsurface drainage is the predominant drainage pathway.

Table 16-2
Suggested Values for Inorganic Constituents
in Wastewater Applied to Land

Potential Problem and Constituent	No Problem	Increasing Problem	Severe
pH (std. units)	6.5 - 8.4		<5.0 >9.0
Permeability			
Electrical Conductivity (mho/cm)	>0.50	<0.50	<0.2
Sodium Adsorption Ratio (a)	<5.0	5.0 - 9.0	>9.0
Salinity			
Electrical Conductivity (mmho/cm)	<0.75	0.75 - 3.0	>3.0
Specific Ion			
Anions:			
Bicarbonate (meq/l)	<1.5	1.5 - 8.5	>8.5
(mg/l as CaCO3)	<150	150 - 850	>850
Chloride (meq/l)	<3.0	>3.0	>10
(mg/l) <100	>100	>350	
Fluoride (mg/l)	<1.8		
Cations:			
Ammonia (mg/l as N)	<5.0	5.0 - 30	>30
Sodium (meq/l)	<3.0	>3.0	>9.0
(mg/l) <70	>70		
Trace Metals (mg/l):			
Aluminum<10			
Arsenic	<0.2		
Beryllium <0.2			
Boron	<0.5	0.5 - 2.0	>2.0
Cadmium	<0.02		
Chromium<0.2			
Cobalt	<0.1		
Copper	<0.4		
Iron	<10		
Lead	<10		
Lithium	<2.5		
Manganese	<0.4		
Molybdenum	<0.02		
Nickel	<0.4		
Selenium <0.04			
Zinc	<4.0		

a Sodium Adsorption Ratio = $$\frac{Na+1}{SQR(\ (Ca+2 + Mg+2)/ 2)}$$

Where, Na+1, Ca+2 and Mg+2 in the wastewater are expressed in milliequivalents per liter (meq/l). SQR represents 'square root of'.